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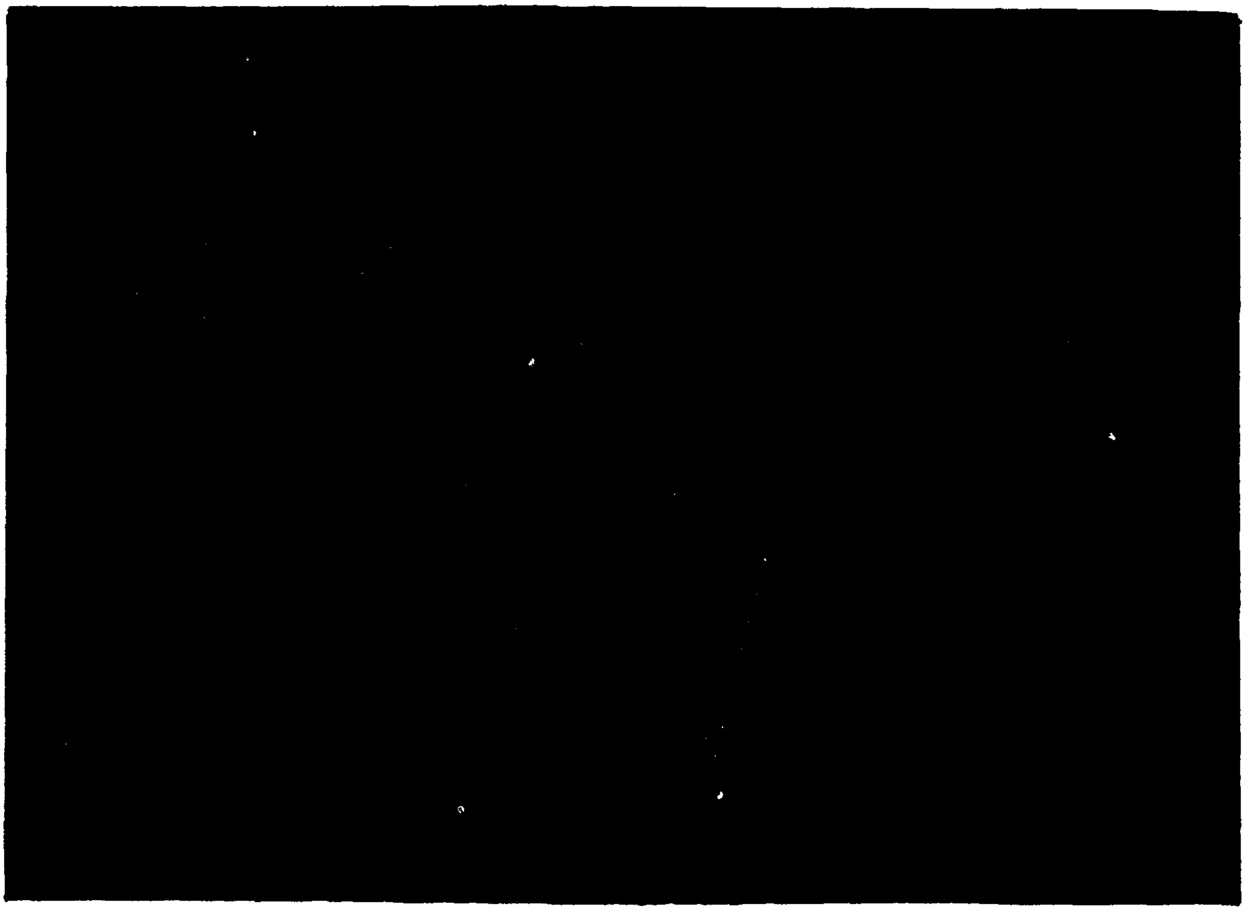
Richard B. Russell Dam and Lake

Savannah River, Georgia And South Carolina

Final Foundation Report
Volume 1 of 2



Overview Of Richard B. Russell Dam And Powerhouse

US Army Corps
of Engineers
Savannah District

Date OCT 0 3 1989

DEPARTMENT OF T: E ARMY
SAVANNAH DISTRICT, CORPS O ENGINEERS
P.O. Box 889
SAVANNAH, GEORG'A 31402-0889

REPLY TO
ATTENTION OF:

CESAS-EN-B 30 November 1989

MEMORANDUM FOR Commander, Cameron Station, Alexandria, VA 22314, ATTN: DTIC/DA-2

SUBJECT: Richard B. Russell Dam and Lake, Savannah River, Georgia and South Carolina, Final Foundation Report

1. Enclosed are twelve (12) copies of referenced report, submitted in accordance with ER 1110-1-1801.

2. If you have any questions, please feel free to call the Project Manager, Mr. Bill Lynch, at 912-944-5701.

FOR THE COMMANDER:

Encls (12 cys)

JOSEPH H. ROGERS, JR., P.E.
Acting Chief, Engineering Division

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

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16. DISTRIBUTION STATEMENT (of this Report)

Distribution of report was in accordance with ER 1110-1-1801

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

The abstract was written for Block 20 of this form which was included in all reports submitted to Defense Technical Information Center (DTIC) according to ER 1110-1-1801

18. SUPPLEMENTARY NOTES

The report was prepared from written naratives, drawings, and oral information given by project geologists Timothy A. Pope and Charles H. Combs.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Explorations and Investigations prior to construction
Regional Geology
Site Geology
Engineering Characteristics of soils and bedrock
Special design considerations (See Reverse Side)

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The report is a detailed description of the work involved in building of the Richard B. Russell Project. This includes the concrete dam, earth embankments, and the powerhouse. The report covers initial site exploration, geology of the site, special design considerations, excavation procedures, foundation exploration during construction, unusual conditions present in the foundation and foundation treatment. Grouting of the embankments and concrete dam is illustrated in numerous plates that show the various grout curtains present. The character of the foundation is shown in photos and foundation maps. The report

19. (Continued)

Excavation Procedures
Foundation Exploration During Construction
Unusual Conditions
Special Fault Zone Treatment
Foundation Anchors and Bolts
Character of Foundation
Foundation Treatment
Grouting
Instrumentation
Possible Future Problems.

20. (Continued)

also describes instrumentation that was installed during and after construction. Possible future problems are also addressed.

RICHARD B. RUSSELL PROJECT

SAVANNAH RIVER, GEORGIA AND SOUTH CAROLINA

FINAL FOUNDATION REPORT

CONCRETE DAM, EMBANKMENTS AND POWERHOUSE

In Two Volumes

VOLUME I - TEXT AND PLATES

U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

RICHARD B. RUSSELL PROJECT

SAVANNAH RIVER, GEORGIA AND SOUTH CAROLINA

FINAL FOUNDATION REPORT

CONCRETE DAM, EMBANKMENTS, AND POWERHOUSE

INDEX

RICHARD B. RUSSELL PROJECT

SAVANNAN RIVER, GEORGIA AND SOUTH CAROLINA

FINAL FOUNDATION REPORT

CONCRETE DAM, EMBANKMENTS, AND POWERHOUSE

CONTENTS

TEXT

CONTENTS (Cont'd)
TEXT (Cont'd)

CONTENTS (Cont'd)
TEXT (Cont'd)

CONTENTS (Cont'd)
TEXT (Cont'd)

CONTENTS (Cont'd)

FIGURES

CONTENTS (Cont'd)

TABLES

CONTENTS (Cont'd)

PLATES

CONTENTS (Cont'd)
PLATES (Cont'd)

CONCRETE DAM (Cont'd):

EMBANKMENTS:

CONTENTS (Cont'd)
PLATES (Cont'd)

EMBANKMENTS (Cont'd):

CONTENTS (Cont'd)
PLATES (Cont'd)

EMBANKMENTS (Cont'd):

CONTENTS (Cont'd)

APPENDICES

(Volume II)

RICHARD B. RUSSELL PROJECT

SAVANNAH RIVER, GEORGIA AND SOUTH CAROLINA

FINAL FOUNDATION REPORT

CONCRETE DAM, EMBANKMENTS, AND POWERHOUSE

PERTINENT DATA

LOCATION OF DAMSITE

Savannah River, Georgia and South Carolina, 275.1 miles above the mouth, 262.0 miles above Savannah, Georgia, 63.0 miles above Augusta, Georgia, and 37.4 miles above Clarks Hill Dam.

DRAINAGE AREA	Square Miles
Above mouth of Savannah River	10,579
Above Augusta, Georgia	7,245
Above Clarks Hill Dam	6,144
Above Richard B. Russell damsite	2,900
Above Hartwell Dam	2,088
Between Richard B. Russell and Hartwell Dams	812

ESTIMATED NATURAL STREAMFLOW AT DAMSITE	c.f.s.
Mean discharge for period of record (31 years)	5,078
Minimum discharge	
Instantaneous (11 May 1961)	300
Daily (24, 25 September 1961)	340
Monthly (May 1961)	969
Maximum discharge	
Instantaneous (25 August 1908)	114,000
Monthly (March 1952)	16,910

DESIGN FLOWS	c.f.s.
Standard project flood	
Peak reservoir inflow	518,000
Maximum estimated outflow	360,000
Spillway design flood	
Peak reservoir inflow	1,035,210
Maximum estimated outflow	801,500

LAKE ELEVATIONS	Feet, m.s.l.
Spillway design flood	490.0
Induced surcharge pool	485.0
Standard project flood	484.3
Flood control pool	480.0

PERTINENT DATA (Cont'd)

LAKE ELEVATIONS (Cont'd)	Feet, m.s.l.
Maximum power pool	475.0
Average power pool	473.0
Minimum power pool	470.0

RESERVOIR AREAS	Acres
Standard project flood, elevation 484.3	31,770
Flood control pool, elevation 484.3	29,340
Maximum power pool, elevation 475	26,653
Minimum power pool, elevation 470	24,117
Limits of clearing, elevation 477 - elevation 465	5,880
Limits of clearing, elevation 465 - elevation 430	11,890
Limits of clearing, below elevation 430	7,540

RESERVOIR STORAGE VOLUMES	Acre-feet
Spillway design flood, elevation 490	1,488,155
Standard project flood, elevation 484.3	1,305,000
Flood control pool, elevation 480	1,166,166
Maximum power pool, elevation 475	1,026,200
Minimum power pool, elevation 470	899,400

ALLOCATED STORAGE VOLUMES	Acre-feet
Flood storage, elevation 475 - elevation 480	140,000
Power storage, usable, elevation 475	126,800
Surcharge storage, elevation 480 - elevation 490	322,000

TAILWATER ELEVATIONS	Feet, m.s.l.
Spillway design flood (801,500 cfs)	356.0
Standard project flood (360,000 cfs)	343.0
Maximum pool of record	331.0
Four units operating at average head	327.5
Maximum static condition	312.0

DAM

Type: Concrete gravity and earth embankment	
Length, feet	
Concrete section	1,883.5
Earth embankments	2,632.0
Saddle dike	1,100.0
Elevations, m.s.l.	
Top of dam	495.0
Streambed	300.0
Spillway crest	436.0
Top of tainter gates, closed	481.0

DAM (Cont'd)

Freeboard, feet	5
Height, maximum, feet	
Concrete section	205
Earth embankment	160
Top width, maximum, feet	
Concrete (nonoverflow)	20.7
Earth embankment	20.0
Side slopes, earth embankment	
Upstream	1V on 3H
Downstream	1V on 2.5H
Spillway	
Type: Concrete gravity ogee	
Gross length, feet	590
Clear opening, length, feet	500
Tainter gates, number, width, heights	10-50'x 45'
Type of bucket	flip
Radius of bucket, feet	50
Bucket lip, elevation m.s.l.	357.25
Intake section	
Length, feet	
Conventional units	284.0
Pumped storage units	315.5
Total	599.5
Intake invert elevation m.s.l.	370.0

POWERPLANT EQUIPMENT

Initial units, number	4
Ultimate units, number	8*
Net operating head, maximum, feet	162
Average operating head, feet	144
Minimum operating head, feet	134
Assumed head losses, feet	1
Turbine capacity per unit, hp, Average head	104,000
Maximum discharge per unit at minimum head, cfs	8,000
Unit spacing, feet	
Conventional units	71
Pump units	80
Transition spacing	75.5
Type of turbine	Francis
Elevation of distributor	300
Elevation bottom of draft tube	
Conventional units	257
Pump units	250
Draft tube exit elevation, conventional and pump units	265
Length of draft tube, feet	71
Generator capacity per unit, kW	75,000
Initial installation, total kW	300,000

POWER PLANT EQUIPMENT (Cont'd)

Ultimate installation, total kW	600,000
Dependable capacity, initial installation, kW	300,000
Generator rating (95 percent pf), kVA	78,947
Generator speed, rpm	120
Generator voltage, kV	13.8
Transformer rating, kVA, Two 3-phase, each	182,000
Transformer voltages, kV	13.2 to 230
Average annual energy, million kWh	
At site	464.5
Prime	365.0

* Provisions have been made for the future installation of four 75,000 kW pump turbine units which will provide an additional 300,000 kW of capacity. An extensive system of rock bolting and rock anchorage was implemented for adding stability against uplift forces acting against the bases of units 5, 6, 7, and 8 (the future motor-generator bays). References to this special anchoring treatment are found throughout this text.

QUANTITIES

Concrete dam (nonoverflow and spillway)	
Excavation, common, cy	70,000
Excavation, rock, cy	105,000
Excavation, dredged, cy	209,000
Mass concrete, cy	562,000
Concrete, spillway piers, and training walls, cy	22,100
Concrete, terminal cone retaining wall	950
Reinforcing steel, lb.	3528,000
Tainter gates and embedded metal, lb.	2475,000
Stoplogs and guides, lb.	324,000
Sluice gates and embedded metal, lb.	290,000
Miscellaneous steel, lb.	18,000
Galvanized steel, lb.	26,000
Concrete dam (power intake section)	
Excavation, common, cy	19,000
Excavation, dredged, cy	212,000
Excavation, rock, cy	95,000
Mass concrete, cy	338,000
Concrete, intake headwork, cy	76,000
Reinforcing steel, lb.	4,446,000
Galvanized steel, lb.	45,000
Intake gates and guides, lb.	1,464,000
Trash racks and guides, lb.	1,483,000
Bulkheads and guides, lb.	440,000

NOTES: (1) All elevations in this report refer to feet above mean sea level.

(2) Period of record August 1896 to present. Maximum discharge at Calhoun Falls, South Carolina, was computed for flood of 25

August 1908 as being 114,000 cfs. Maximum stage known since at least 1896, that of 25 August 1908, see U.S.G.S. Surface Water Records of Georgia 1969.

1.0 INTRODUCTION

1.1 Location and Description of Project: The Richard B. Russell Dam is located on the Savannah River near Calhoun Falls, S.C., 63 miles above Augusta, Georgia. It is 37.4 miles above Clarks Hill Dam and 29.9 miles below Hartwell Dam (see Figure 1). The Richard B. Russell Dam and Lake is a multipurpose project designed to provide hydropower, some flood control, recreation, and has a potential for water supply. The dam consists of a 195 foot high, 1,884 foot long concrete gravity structure (approximately 1.1 million cubic yards of concrete in 32 monolithic segments) flanked by two earth embankments (approximately 2.9 million cubic yards of zoned material.) The Georgia embankment is 2,180 feet long and the South Carolina embankment is 460 feet long. Within the powerhouse are four 75-megawatt conventional units and space for four 75-megawatt pump units. The project is designed as a peaking plant, with an installed capacity of 600 megawatts.

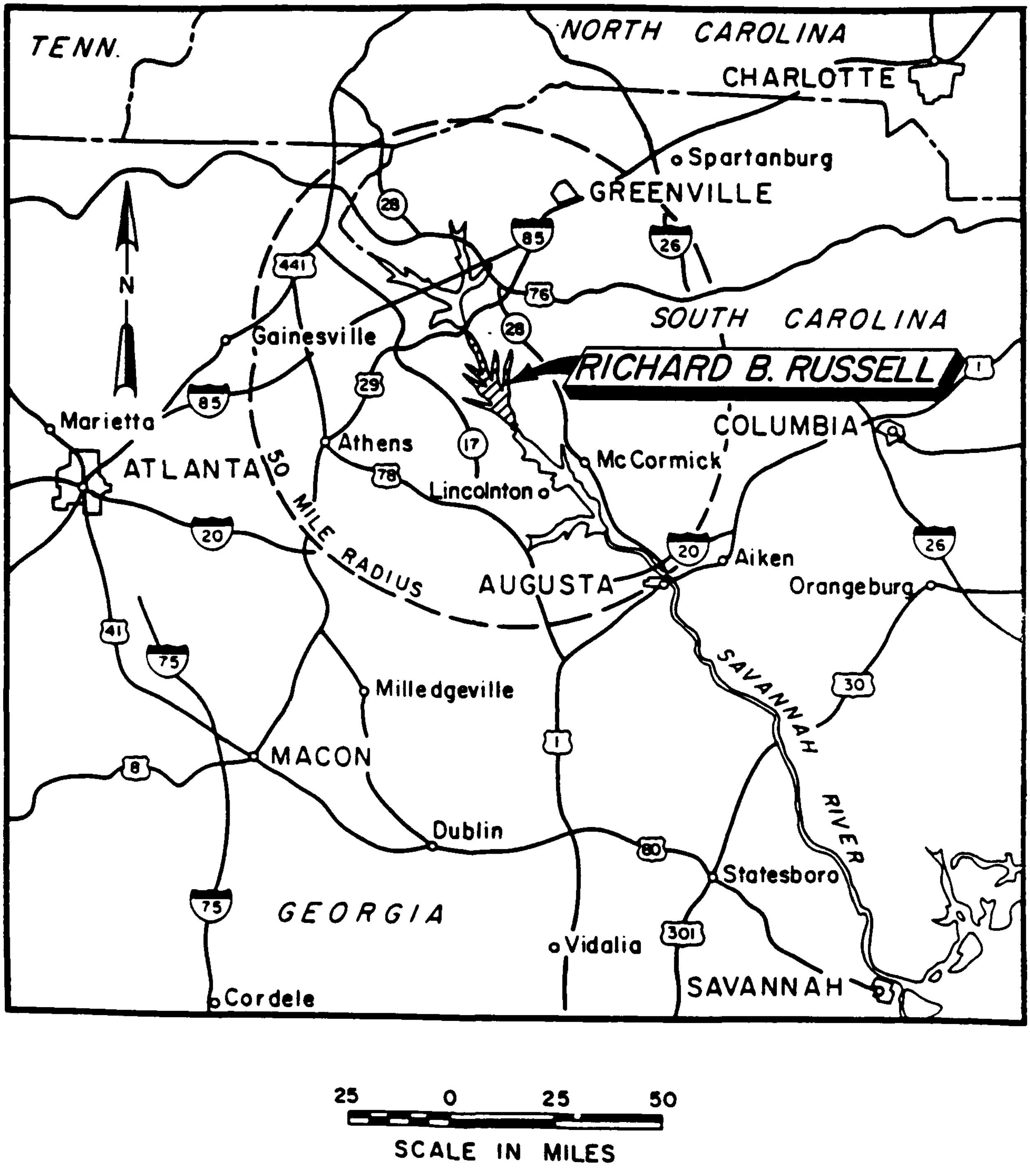
A general plan of the project is shown on Plate 3. The project plan provides for a maximum power pool at elevation 470 feet; spillway design flood pool is elevation 490. Tailwater elevations are: spillway design flood, 356; standard project flood, 343; and average operation (four units at average head), 327.5. The top elevation of both the concrete structure and the earth embankments is 495.

The concrete dam includes a spillway section containing ten tainter gates, an intake section, and a powerhouse containing the aforementioned 75 megawatt units.

The embankments are of rolled-fill construction. The Georgia (right) embankment extends 2,180 feet in a southwesterly direction. The South Carolina (left) embankment trends in an easterly direction for a distance of approximately 460 feet. Both of the embankments tie-in to the adjacent terrain (valley slopes) at elevation 495. Also located on the South Carolina (east) side of the dam is a 1,100 foot long saddle dike. Of similar construction to the earth embankments, the saddle dike was used to fill a swale or low-lying area of South Carolina State Road 269.

In addition to the generation and water retention structures, there were several highway and railroad bridges constructed for the project. Bridges which were founded on rock include the R-1 bridge, where the SCL railroad crosses the Savannah River, the Highway 72 bridge between Georgia and South Carolina, and the Georgia 368 - South Carolina 184 bridge near Iva, South Carolina.

1.2 Construction Authority: Basic project authorization is contained in the Flood Control Act of 1966 - Public Law 89-789, 89th Congress HR 18233, 7 November 1966. Authorization to include minimum provisions for the future installations of pumped storage facilities was granted by the Assistant Secretry of the Army in January 1977. Section 182a of the Water Resources Development Act of 1976 amended the authorization for the Richard B. Russell project contained in Section 203 of the 1966 Act by deleting the phrase which precluded the inclusion of pumped storage power as a part of this project and as a result design proceeded based on providing installation of pumped storage. House Document No. 97-35 dated March 19, 1981 was presented to the 1st Session of the 97th Congress by the Assistant



LOCATION OF RICHARD B. RUSSELL
DAM & LAKE

FIGURE 1

Secretary of the Army. This report was furnished to Congress to advise them of the Chief of Engineers' analysis and intention of proceeding with the construction of pumped storage hydropower facilities.

1.3 Purposes of Report: The purpose of this report is to provide a summary record of the actual geological conditions encountered during construction of the dam, powerhouse, and embankment foundations; the geotechnical methods and processes utilized to insure adequate foundations; and of any conditions which might lead to problems in the future.

1.4 Project History: Construction of the project began with the award of the contract for construction of the west access road to the A.J. Kellos Company in November of 1974. This road, which connects the Bobby Brown Park road to the damsite, was completed in 1975.

In December 1975, Southeastern Highway Contracting Company was awarded the contract for first stage diversion of the Savannah River. They were to excavate a channel into rock west of the intended concrete damsite, construct a bridge across the channel, build cofferdams upstream and downstream of the site, and divert the river through the new channel. During excavation, work was slowed by discovery of a prominent fault which crosses the west embankment area. This fault is described in detail in the Design Earthquake Report (1977) and will also be covered in Section 7.1-EE of this report. First stage diversion was accomplished in Spring 1977.

In August 1977, Lane Construction Corporation of Meriden, Connecticut was awarded the contract for excavation of the concrete dam and powerhouse area. In order to expedite the construction schedule, this phase of the work had been left as a separate contract prior to setting of powerhouse grade and completion of concrete dam design. Excavation began on the east abutment (Monoliths 27-32), then on the west abutment (Monoliths 1-7), then into the intake (Blocks 8-15) and spillway (Blocks 16-26) areas as work was being completed on the east abutment. Excavation in the powerhouse area was being completed as the concrete dam contractor was beginning his trestle construction.

Final elevations in several areas of the excavation were revised as construction progressed. These areas included Monoliths 2 and 3 (where a shallow pocket of decomposed rock was removed with a backhoe), Blocks 16-21, (underlain by a relief joint filled with decomposed rock materials 3 to 5 feet below design grade), and Blocks 26-30, where clay-filled joints and a wide shear zone affected rock quality. Monoliths 27 and 28 required the most extreme lowering, being excavated 15 to 38 feet below original design grade. The work on these two blocks was completed under modification after excavation of the powerhouse area, in the summer of 1979. These changes are depicted on Plates 13 and 16.

The concrete dam contract was awarded to Dravo-Groves (a joint venture) in May of 1978. Preliminary site work began soon after the notice to proceed was issued on 16 June 1978. The batch plant (four, 4.5 cubic yard mixers) was built just west of the erection bay. Aggregate handling facilities were built (some blasting was necessary) in the present switchyard area, and a trestle system was begun in the Geogia abutment area with footers 85 and 121 feet downstream of the dam axis.

Suitable material for aggregate (for the most part identical to the foundation rock types) was located north of the S.C. 269 saddledike location, less than a mile northeast of the dam. The quarry was developed and tested in the fall and winter of 1978 and aggregate production began in January 1979. The original crusher setup, which depended on a roller crusher, was inefficient and had to be replaced in the middle of construction. This change is described in the Concrete Report.

By March 1979, the trestle system had been extended into the intake blocks, a few monolith foundations were being prepared, and the batch plant was ready to produce mass concrete. Where foundation relief varied by more than 5 or 6 feet, "plug pours" were usually made to level-up the placement area. The first mass concrete in the dam structure area was placed in the Monolith 2 plug pour on 19 March 1979. The last placement on a rock surface was completed in Block 32 on 15 February 1981.

Drilling and grouting operations for the concrete dam foundation began in December 1980, and continued through October 1981. A single line, 3 zone curtain, 120 feet deep having holes on 5-foot centers was installed. Some split-spaced holes between those centered every 5 feet were also grouted. Problems which complicated or delayed this work were: leaks through the concrete onto the gallery floor, leaks through the concrete dam into the earth embankment areas, and interception of the inverted plumb bob in Monolith 26 with a grout hole. The contractor experimented with a down-the-hole percussion type hammer in drilling the foundation drain holes, but when this proved inefficient, he completed the remainder with rotary equipment. The rock (especially the metadacite) proved to be very hard to drill, making plug bits less economical than coring bits. The subcontractor (Boyles Brothers) experimented extensively with different types of bits, finally settling on impregnated coring bits. Refer to Plate 50 for details of drain and grout hole drilling.

Instrumentation was installed throughout the job. Difficulty was experienced in getting drills positioned for drilling inverted plumb bob, uplift cell, and deflectometer holes due to the uneven foundations. These holes (with the exception of the deflectometer holes) were drilled by subcontractors, making scheduling a problem for the contractor. If they were drilled before foundation work, rock removal could cut as much as 5 feet from the original drilled depth. If the foundation was prepared first, drilling could delay concrete placement. Finally, the contractor decided to do some rock removal in the general area of the holes prior to drilling them. Instrumentation that was installed included 33 uplift cells in Block 7, 10 cells in Block 10, 33 in Block 23, 10 in Block 26, 19 in Block 27, 23 in Block 28, and 6 in Block 29; plumb bobs and inverted plumb bobs in Monoliths 16 and 26, and deflectometers (1 each) in Monoliths 27 and 28. The deflectometers were installed in the foundation with a stub-up to gallery level. Then concrete was placed to full height with a 6-inch blockout around the deflectometer location. After this, the deflectometer was assembled and installed from the top of the dam, connected to the stub-up in an opening left in the gallery for this purpose, and the block-out grouted up. All work in the concrete dam was considered essentially complete in June 1982.

1.5 Location of Structure: The concrete dam is positioned perpindicularly across the Savannah River channel. Blocks 1-9 are set into the old Georgia riverbank, and Blocks 27-32 are set into a bluff which was the old South Carolina riverbank. Intake Blocks 8-16 and spillway Blocks 16-26 span the old riverbed. The powerhouse is located immediately downstream (south) of the intake area. The concrete structures are situated between the Georgia and South Carolina earth embankments, both of which are founded well above the old riverbed elevation. The earth embankments trend generally in a northeasterly-southwesterly direction, whereas the concrete structures follow a nearly east-west alignment. The deepest part of the river channel was in the vicinity of Blocks 20-22. These spatial relationships are depicted on Plate 3.

1.6 Contractors, Supervision, and Quality Control Organizations: The A.J. Kellos Company built the West Access Road. The Project Manager/Job Superintendent was Mr. McCord.

The contractor on the first stage diversion contract was Southeastern Highway Contracting Company of Gainesville, Georgia. James E. Trammel served as Project Manager/Job Superintendent.

Lane Corporation of Meriden, Connecticut was the contractor for the concrete dam and powerhouse excavation contract. The Project Manager/Job Superintendent was Mr. Darrel Emig. The Quality Control Organization consisted of the Quality Control Chief, Mr. Robert Halpin, and members of the supervisory staff, primarily Mr. H.W. Anderson and Neil Armstrong.

The concrete dam, as well as the earth embankments, were built by Dravo-Groves, (a joint venture of the Dravo Corporation of Pittsburg, Pennsylvania and S.J. Groves and Sons of Minneapolis, Minnesota). Excavation of the cut-off trench beneath the earthen portions of the dam was also handled under this arrangement. The Dravo Corporation was responsible for supervision and operation of the construction forces. The project manager was Mr. Joseph J. Smith. The Job Superintendent was Charlie Ramsey. The Quality Control Organization was composed of the Quality Control Chief, Doug Sickle, and members of the contractor's and subcontractor's supervisory staff. Foundation and instrumentation work were monitored by Mr. Sickle, J.C. Wallis, Jim Adkins, and Roy Stone of Dravo. Grouting and all drilling operations were monitored by Steve Hall of Boyles Brothers (subcontractors). Installation of instrumentation in the dam was monitored by Andy Anderson of Martyn Brothers (subcontractor).

The joint venture of Replublic-Crowder was awarded the powerhouse construction contract in August of 1981. Some modification to the previously excavated foundation surface was also handled by this concern.

1.7 Key Resident and Design Staff:

Key Resident Staff:	Resident/Area Engineer	Jim B . Lloyd	1976-1982
	Asst. Resident Engineer	Clyde R. Orr	1977-1979
	Asst. Area Engineer	James E. Evans	1979-1982
	Project Geologists	Robert H. Stephens	1976-1977
		James E. Hastings	1977-1978

	Project Geologist	Timothy A. Pope	1979-1982
	Staff Geologists	Timothy A. Pope	1978-1979
		Mary Lou Fagan	1979-1981
		Charles H. Combs	1981-1982
	Technicians	Jack Ford	1978
		Thomas B. Keane	1979-1980
		James R. Zanders	1980-1982
Key Design Staff:	Geologists	Robert S. Stansfield	1976-1979
		Charles M. Deaver	1974-1979
		Earl F. Titcomb, Jr.	1961-1985
		Ron M. Rhodes	1978-1979
	Engineers	Ben C. Foreman	1974-1984
		Tom Durrence	1982
		John Hager	1984

2.0-CD CONCRETE DAM FOUNDATION EXPLORATIONS (PRIOR TO CONSTRUCTION)

A number of subsurface exploratory programs were carried out prior to construction. Among these were:

2.1-CD Site Selection: Borings A-1 through A-18: These borings were located along a baseline designated by the letter "A" which at one time was being considered as a potential axis for the concrete dam. Drilling of these borings was carried out between February and June of 1968.

Borings B-2, B-4, B-5, B-6, B-7, B-8, B-10, B-12, B-13, and B-15: These borings were located along a baseline designated by the letter "B" which also was under consideration as a possible dam axis. Drilling of these "B" prefixed borings occurred between March and May of 1968.

Borings C-1 through C-24: These borings were taken along a baseline designated by the letter "C". Drilling was conducted between March and June of 1968. Baseline "C", with slight modification, ultimately became used as the concrete dam axis.

Borings 1, 2, 3, and 4 were also drilled in conjunction with the site selection process and their locations were relative to baseline "C". These were completed during October and November 1964.

Borings PF-1, PF-2, PF-3, PF-3A, PF-4, PF-6, PF-7, and PF-8 were drilled relative to baseline "A" in November and December 1961.

Borings PQ-1, PQ-2, PQ-3, and PQ-4 were drilled between November and December of 1961 in an investigation of potential quarry sites for rock aggregate.

Geophysical testing such as cross-hole sonic velocity measurement and Schmidt Hammer rebound testing was also performed, the results of which are presented under Section 3.5: "Engineering Characteristics of the Bedrock Materials".

Plate 4 shows the linear arrangements of the potential baselines once considered for the concrete dam.

2.2-CD General Design: Following selection of baseline "C" as the approximate location of the dam axis, a number of additional borings were made, the locations of which were relative to this baseline. Borings C-25 through C-96, drilled between September of 1968 and August of 1969 are among these.

Other investigations, which concern the embankments, saddle dike, service-road fill, powerhouse, and potential borrow areas will be highlighted later in this text.

2.0-EE EARTH EMBANKMENT EXPLORATIONS (PRIOR TO CONSTRUCTION):

2.1-EE Foundation Explorations: The foundations for the Georgia and South Carolina embankments were explored by 90 and 51 borings, respectively. Also, on the South Carolina side, 11 additional borings were drilled in the

area of the service road embankment and 7 borings drilled along the axis of the saddle dike. Locations of all borings made for the Georgia and South Carolina embankment foundations are among those shown on Plates 58 and 59, respectively. The majority of the borings were continuously split-spooned through the overburden to refusal and then core drilled to obtain either 2.5-inch or 4-inch core samples. Selected 3 and 5-inch Shelby tube undisturbed samples were obtained from the overburden soil for laboratory testing. Selected 4-inch core samples were obtained of the saprolite (intensely weathered rock) and wrapped in cheesecloth and coated with wax to preserve them for laboratory consolidation and triaxial testing. Of the core borings, 43 were hydraulically pressure tested. In addition, the deep diversion channel excavation crossing the proposed alignment of the Georgia embankment served as an excellent inspection trench by exposing to view some of the rock types, geologic structures, and rock weathering that would be encountered in the cutoff trench.

2.2-EE Materials Explorations: A total of 182 borings and 15 backhoe test pits were made in the search for impervious borrow. Borings were made by hand auger, standard split-spoon, "square" or Vicksburg type auger, and spiral auger. "Four-by-five" double-tube core borings were made in attempts to locate intensely weathered rock. Soil-type borrow material was sampled with the hand auger, the split-spoon sampler, and "square" auger. The intensely weathered rock (saprolite) was sampled with spiral augers to determine the probable depths to which ripping could be accomplished. From the explorations conducted for the excavated diversion channel it was found that the spiral auger sampling provided a fairly good indication as to the depths of rippable material. Jar samples were taken in the borings for determination of in-situ moisture content, and bulk samples were obtained for laboratory testing. The river sand was investigated by standard split-spoon borings. Furthermore, an extensive test fill program was conducted for the impervious soil borrow and the sand in the dredged sand stockpile.

2.3-EE Groundwater Investigations: Groundwater levels recorded in exploratory borings showed that the groundwater at the site were effluent to the Savannah River. On the Georgia side, the water table was found to be above, or coincident with, the top of rock. On the South Carolina side, where the topography is steeper, the water table was generally found to be coincident with or below the top of rock.

2.4-EE Laboratory Investigations: Jar and bulk samples from the proposed impervious borrow area, bulk samples of the intensely weathered rock and dredged sand, and jar and undisturbed samples from the embankment foundations were tested in the South Atlantic Division Laboratory. Mechanical and hydrometer analysis, moisture content, and Atterberg limits were performed on the jar samples. In addition to the above tests, the following tests were performed on bulk samples of impervious and intensely weathered rock: standard compaction, consolidation, permeability, direct shear, and triaxial compression. Additionally, x-ray diffraction, pinhole erosion and Soil Conservation Service laboratory dispersion tests were performed on the impervious borrow material. Tests on the dredged sand included mechanical analysis, specific gravity, standard and modified compaction tests, maximum-minimum density tests, permeability, direct shear, and triaxial compression.

2.5-EE Dynamic Analysis: The proposed embankments were subjected to a dynamic stability analysis performed by the Waterways Experiment Station utilizing results from exploratory drilling, sampling, and field geophysical studies. Undisturbed samples were obtained from the foundation residual soil and saprolite and from test fills constructed of impervious material and dredged sand for appropriate dynamic testing. Bulk samples of the impervious, intensely weathered rock, and dredged sand were also obtained for appropriate dynamic testing. Dynamic testing of the materials was conducted at the South Atlantic Division Laboratory.

2.6-EE Clay Seams Study: The excavation for the concrete dam in the South Carolina abutment exposed mud and clay-filled seams in the rock. The seams first appeared as the excavation was approaching initial design grade, and it appeared that seams might be found below grade. Through the use of the exploratory borings and field inspections of the excavation as the monoliths reached design grade, the problem of the clay seams was carefully evaluated and addressed to ensure adequate and safe foundations for the concrete structure. Based on extensive field and laboratory investigations, it was concluded that the clay seams were not very continuous and did not pose a threat to the integrity and safety of the earth embankment. Because of the presence of the clay seams, the possibility of a portion of the abutment immediately upstream of the embankment sliding into the reservoir, and creating a wave which might overtop the dam was evaluated. But considering the facts regarding the clay seams, and since the abutment will be inundated by the reservoir, such an occurence was considered a near impossibility. However, because there appeared to be a considerable number of seams in certain areas of the steeply dipping foundation for the downstream terminal cone, stability analyses were performed for the terminal cone assuming clay seams were completely continuous through the foundation. The analyses reinforced the conclusion that the clay seams pose no threat to the embankment.

2.7-EE Filter Tests: Laboratory filter tests were conducted by WES to establish the adequacy of the proposed sand as a filter, both in regard to its internal stability and its effectiveness in protecting the impervious core material. For conservatism, these tests utilized some of the finest core material and some of the coarsest sand. The tests indicated that stockpiled sand would adequately protect and filter the impervious core material.

2.0-PH POWERHOUSE FOUNDATION EXPLORATIONS (PRIOR TO CONSTRUCTION)

2.1-PH Previous Investigations: Geologic investigations made prior to construction are described fully in Design Memorandum 8 - Geology, General Design Memorandum 1, and Design Memorandum 12 - Materials. Geologic conditions of the project are described in detail in Section 3.0 of this report, and summaries of the geologic/geophysical testing programs may be found in Section 3.5 and Appendix A of this report. The 1977 COE report entitled "Geological Seismological Evaluation of Earthquake Hazards at the Richard B. Russell Project" covers in detail the regional and local seismicity of the project area and an evaluation thereof.

3.0 GEOLOGY

3.1 Regional Geology: The Richard B. Russell Project is in the Piedmont Physiographic Province. The area is broadly rolling with generally long, rounded stream divides. In the Southeastern Piedmont, the country rocks are commonly interbedded, metamorphosed, volcanic, and sedimentary units intruded by dikes and sills. After deposition, these rock layers were folded, faulted, and metamorphosed in several tectonic and metamorphic events. Plutons and dikes intruded the rock mass after metamorphism.

The lower Piedmont in Georgia and South Carolina can best be characterized as a series of NE-SW trending bands of alternating high and low grade metamorphic rocks (see Figure 2). Folding in these rocks is complex and variable with major fold axes trending NE-SW. The lower grade bands are usually metamorphosed to greenschist grade, with well-preserved depositional features common in sedimentary and some pyroclastic rocks. The low-grade bands are called the Carolina Slate Belt and Kings Mountain Belt. The Little River Series of Georgia (type locality is the Little River area on Clarks Hill Lake) is a continuation of the Carolina Slate Belt. The higher-graded bands (usually amphibolite grade) are referred to as Charlotte Belt rocks. In these rocks, preserved depositional features are uncommon, with metamorphic foliation the dominant fabric in pelitic rocks.

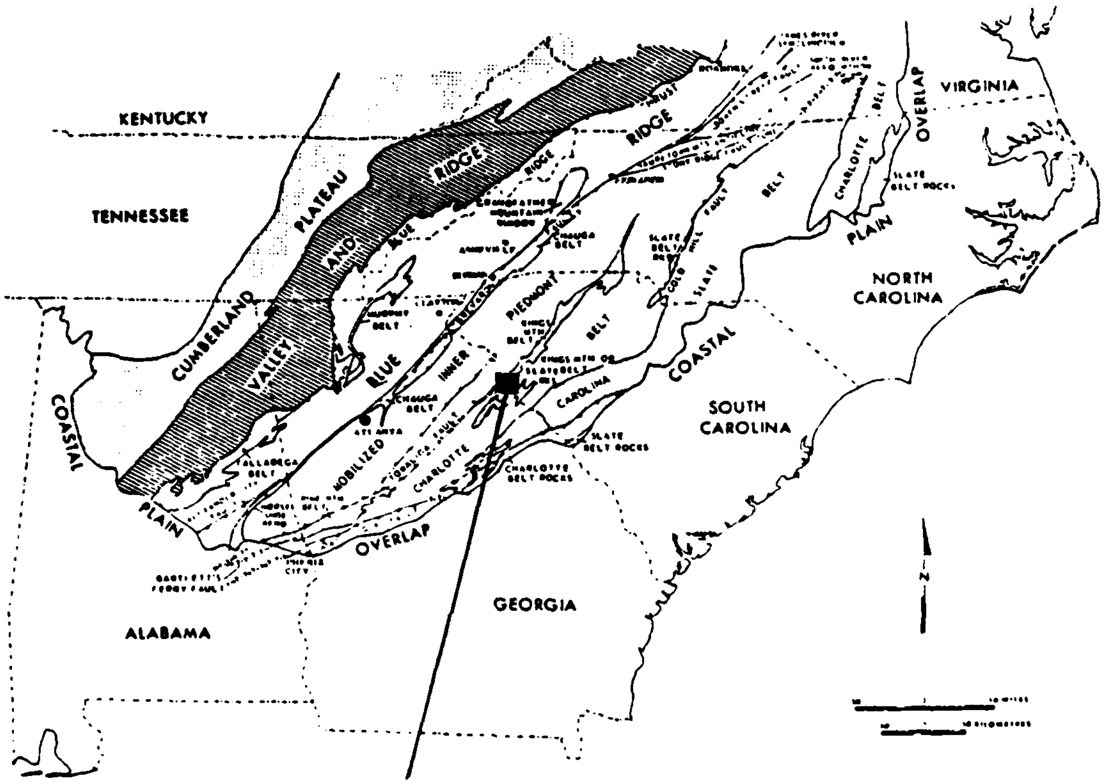
Faulting in the region was summarized in the Design Earthquake Report (COE, 1977) and Table 1 lists information on the faults of major interest. Some of these are shown in Figure 3 and none of them is considered to be active. Due to the presence of faults and location of the project within 170 miles of the Charleston earthquake of 1886 and occurrences of minor tremors in the vicinity, the dam was designed to withstand an earthquake of 5.5 Richter force with a shallow hypocenter at the damsite. The seismic-defensive design resulted in more complex zonation in the concrete edifice, a conservative approach to foundation preparation, and increased instrumentation.

3.2 Site Geology:

a. Physiography: The Savannah River Basin is long and relatively narrow. The Richard B. Russell Lake extends from the dam approximately 29 miles upstream, almost to Hartwell Dam. Maximum width at any point is approximately 3 miles.

The concrete dam is located in a bend of the river in the head waters of Clarks Hill Lake. The river width at the damsite is approximately 1000 feet at elevation 330 MSL. The Georgia riverbank is somewhat lower than that in South Carolina, necessitating a 2180-foot long west embankment. The South Carolina abutment of the concrete dam is located on a high bluff with only a 460-foot embankment needed to impound the lake. Maximum relief at the site (above the old riverbed) is about 250 feet. The lake is approximately 175 feet deep at its deepest point.

b. Description of Overburden: The Richard B. Russell area typically has a normally developed soil horizon, but in some areas the A horizon is absent and the B horizon attenuated. Residual soils vary in color from yellow or tan to dark red. The most common residual soil type is a brick



TECTONIC ZONES IN THE PIEDMONT

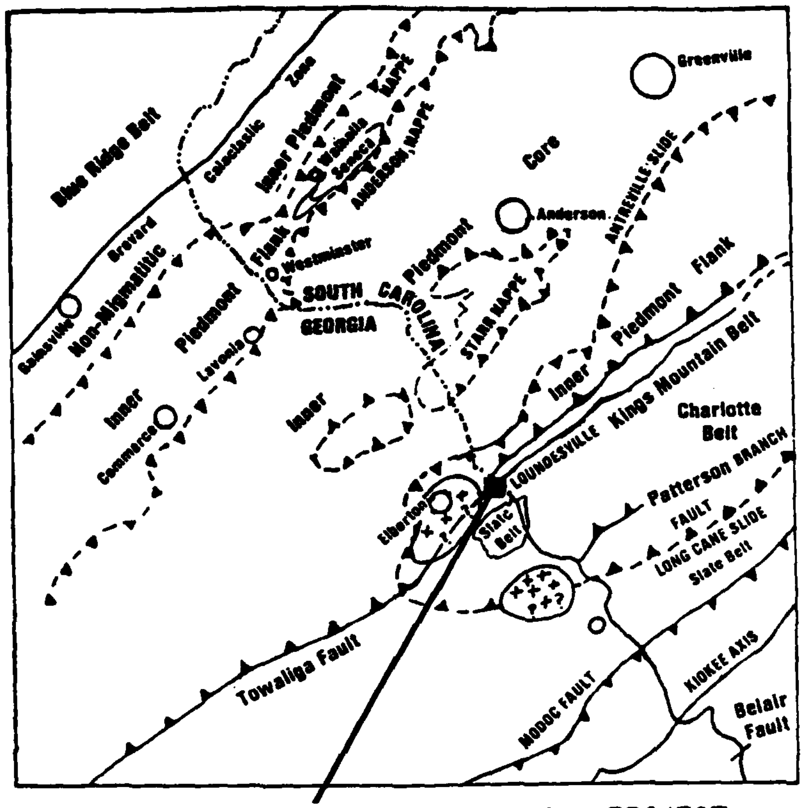
ADAPTED FROM HATCHER (1972)

FIGURE 2

TABLE 1

FAULTS STUDIED IN THE DESIGN EARTHQUAKE REPORT

FAULT	MINIMUM MAPPED LENGTH (MI)	FAULT TYPE	PROBABLE AGE (MAX)	PROBABLE AGE (MIN)	STRIKE	APPROX LOCATION FROM DAMSITE
Modoc	79	1. Thrust 2. Strike-slip	Paleozoic	Late Devonian to Permian	NE-SW	30 miles SE
Lowndesville	160	Questionable Right Lateral Strike-Slip	Paleozoic	Late Devonian	NE-SW	15 miles NW
Brevard	450	1. Right Lateral Strike-Slip 2. Dominantly Dip-Slip	Paleozoic	Late Devonian	NE-SW	75 miles NW
Belair Fault	13	Reverse	50 M.Y.B.P.	Quaternary	NE-SW	50 miles SE-West of Augusta, GA
Patterson Branch	4.5	Normal or Thrust		Mesozoic	NE-SW	McCormick Co, SC approx 12 mi SE
Diversion Channel Fault	4/10	Thrust	355 M.Y.B.P	270 M.Y.B.P	N-S	Under west embankment



RICHARD B. RUSSELL PROJECT

TECTONIC MAP OF THE PROJECT AREA

ADAPTED FROM C.O.E. DESIGN EARTHQUAKE REPORT (1977) FIGURE 3

red, readily erodible silt with a variable sand component. Most samples taken during exploration were classified as SM, SC, ML, or CL. Silica hardpan (formed from a weathering product of feldspars) can be seen in the area, but is usually poorly developed compared to that occurring in the Lincolnton, Georgia area.

Alluvium is confined primarily to the riverbed and its margin. River sand is normally a fairly clean sand composed mostly of subrounded quartz particles with some mica flakes and feldspar fragments. Sand and rounded cobbles in near vertical, open joints caused problems during drilling and blasting operations in the tailrace area.

Hard rock outcrops are rare in the region and, when found, are usually located along streams. Rock is commonly covered by 25 to 100 feet of overburden. Most of this thickness is saprolite occurring in the C and D soil horizons. Due to the rapidity of weathering in the southeast, this material shows relict structure which can often be used to identify the parent rock.

c. Bedrock Stratigraphy: The Richard B. Russell Dam is located in an isolated outlier of low-grade metamorphic rocks within the Charlotte Belt. The rocks have been referred to variously as Carolina Slate Belt, Kings Mountain Belt, and Little River Series lithologies. Rocks mapped in the concrete dam foundation are listed in the following paragraphs in order of probable age of emplacement. These rocks were identified in hand sample with the help of previous microscopic petrographic reports done for design memoranda and reports from a master's thesis done at the University of South Carolina by Weisenfluh (1977). Figure 4 provides further information about stratigraphic relationships of the area.

Xenoliths - A few xenoliths were mapped within the metadacite, mostly on the South Carolina side of the river. They are mafic-intermediate, fine-grained, olive colored rocks with some dark green, corroded phenocrysts of indeterminate mineralogy. They may represent the original rock mass intruded by the metadacite. The rock type is probably meta-andesite.

Metadacite - This is the major rock type found in the dam foundation area. It is a felsic (light-colored with quartz and feldspar as the major constituents) rock, generally of medium to coarse grained, with bluish quartz and cream-colored feldspar making up 85-90% of most samples. Accessory minerals are chlorite and/or biotite with small amounts of epidote, calcite, magnetite, pyrrhotite, and other opaques. It is indistinguishable in hand sample from the Lincolnton metadacite, which occurs about 20 miles south of the damsite. This rock, considered to be the oldest foundation rock, is referred to as quartz feldspar gneiss on maps and core logs.

Quartz Feldspar Porphyry - A fine-grained, dark gray rock with larger (up to 1.6 cm), subrounded phenocrysts of light green, saussuritized plagioclase feldspar and slightly opalescent, bluish quartz occurs in elongate bodies within the metadacite country rock, sometimes bordered by metadiabase dikes. On maps, this rock is called quartz feldspar porphyry. Weisenfluh (1977) refers to this rock as a quartz keratophyre (Na+ metasomatized, submarine, volcanic intrusion). He describes the mineralogy

STRATIGRAPHIC RELATIONSHIPS

ERA	PERIOD		AGE AT BASE OF PERIOD, MY	PIEDMONT: ROCK SEQUENCES: GEORGIA	PIEDMONT: ROCK SEQUENCES: SOUTH CAROLINA	PIEDMONT: INTRUSIVE SEQUENCE	PIEDMONT: METAMORPHIC EVENTS	OUTSIDE 50 MILE RADIUS OF DAM
CENOZOIC	TERTIARY		65					
MESOZOIC	CRETACEOUS		136	TUSCALOOSA FORMATION	TUSCALOOSA FORMATION	FLINTY CRUSHED ROCK		
	JURASSIC		195					
	TRIASSIC		225	GRABEN BASIN FILL	GRABEN BASIN FILL	DIABASE DIKES		
PALEOZOIC	PERMIAN						METAMORPHIC EVENT	
	PENNSYLVANIAN		280					
	MISSISSIPPIAN		345	RHYOLITE FLOWS AND DIKES				
	DEVONIAN	LATE						
		MIDDLE						
		EARLY	395			GOSHEN GRANITE	METAMORPHIC EVENT	
	SILURIAN		437	WACKE CONGLOMERATE SEQUENCE		GABBRO, DIORITE SYENITE COMPLEXES ELBERT GRANITE		
	ORDOVICIAN	LATE					METAMORPHIC AND DEFORMATIONAL EVENT	
		MIDDLE						
		EARLY	500	LITTLE RIVER SERIES: UPPER SEDIMENTARY SEQUENCE	BELAIR BELT	MEDIUM GRAINED ADAMELLITE		
	CAMBRIAN	LATE						
		MIDDLE		LITTLE RIVER SERIES: FELSIC PYROCLASTIC SEQUENCE				
		EARLY	570	LITTLE RIVER SERIES: LINCOLNTON METADACITE				
PRECAMBRIAN	LATE			KINGS MOUNTAIN (WACOOCHEE) BELT; DADEVILLE BELT; INNER PIEDMONT	LOWNDESVILLE; KINGS MOUNTAIN BELT; INNER PIEDMONT	SERPENTINE & ULTRABASIC TABULAR BODIES		
	MID		[illegible]	UCHEE BELT; KIOKEE BELT	CHARLOTTE BELT; KIOKEE BELT	COARSE GRAINED ADAMELLITE STOCKS	INTENSE METAMORPHIC EVENT	

Outside 50 mile radius of dam: VALLEY AND RIDGE (Jurassic to Cambrian); BLUE RIDGE (Devonian to Late Precambrian); CHAUGA BELT (Early Cambrian to Late Precambrian)

ADAPTED FROM C.O.E. DESIGN EARTHQUAKE REPORT (1977)

FIGURE 4

of the microcrystalline ground mass as quartzo-feldspathic, with sericite mica and accessory epidote, sphene, biotite, and chlorite. Field relationships indicate that this rock intrudes the metadacite and is cut by metadiabase dikes.

Mylonite - A very fine-grained, dark gray rock which may be primarily quartzo-feldspathic in composition (minor weathering produces a white rind) was found in the spillway foundation and in Blocks 25-27. The dark color is probably due to the fineness of the translucent grains. This rock occurs in near vertical, tabular bodies cut by dark green metadiabase dikes. Foliation is evident in some parts. This rock was interpreted in the field, without microscopic study, as ancient mylonite. It may be older or younger than the quartz feldspar porphyry.

Metadiabase (or Metabasalt) - This is a mafic (darker in color with an abundance of iron- and magnesium-rich minerals) rock, generally of fine to medium grain size. It is a metamorphosed diabase or basalt, and is referred to on maps and logs as epidote hornblende gneiss. In some cases, this rock may not contain enough epidote to warrant the use of the mineral in the rock name, but this is not discernable in hand sample. Hornblende and feldspar are the major minerals in the rock with minor epidote, chlorite, quartz, pyrrhotite, magnetite, and pyrite.

Variations in the epidote hornblende gneiss should be mentioned. Petrographic reports on this rock show variable mineralogy, and cross-cutting relationships indicate that there are several generations of dike intrusion. The material varies in color from dark green to black. Some dikes, especially thin ones and those parts of thicker dikes which have been sheared, show pronounced foliation, probably indicating a higher chlorite content. The term "chloritic epidote hornblende gneiss" has been used on the geologic maps in this report to indicate this type of rock. The NNW-SSE trending dike in Monoliths 5 and 6 is a green-black, fine-grained material with quartzo-feldspathic inclusions which have been partially recrystallized, possibly from the intense heat of the intruding mafic melt. Some of the inclusions had alternating light and dark gray beds or bands. Another variation in the type of dike mapped is epidote hornblende gneiss with phenocrysts of feldspar, quartz, or hornblende. The quartz phenocrysts in these dikes are similar in appearance to those in the metadacite and quartz porphyry, possibly indicating hybridization of the intruding magma. These dikes cut and were cut by metadiabase dikes (with no phenocrysts) which were indistinguishable from each other in hand sample.

d. Bedrock Structure, Jointing, Discontinuities: Structural features in the dam foundation include several types of faults. The most common is a simple, clean break along which there is a small displacement. Due to the limited displacement on these faults, it is difficult to tell if they are strike or dip-slip faults, but some indicate both types of movement. These breaks are near vertical, commonly trending NW-SE, and are of limited extent. Similar, but longer faults occur in Blocks 2 through 7. These faults (partially lined with epidote) trend NE-SW, have variable, near vertical dips, and show slickensides with well-developed striations parallel to the dip direction. Low angle faults, healed by zoned quartz and chlorite veins, also occur in the foundation. These are apparently reverse faults of

35 degrees or less dip. They offset some metadiabase dikes and are themselves offset by (visually) identical dikes.

Planes and zones of shear were also mapped. These zones vary from less than an inch to more than a foot thick. They commonly cut along dikes, but are also found sheared through the country rock for short distances. In addition to crushed and altered rock, these zones contain chlorite, calcite, a pink zeolite, gouge, and other materials of hydrothermal origin. Sense of movement along these shear zones is commonly indeterminate, with evidence of multiple movement direction seen in several zones where slickensides and "stairstep" breaks would allow determination of past movement. The largest of these zones cuts through a subparallel dike swarm in Blocks 26 through 29. Slickensided breaks in this zone were found in Monolith 28. They trended NE-SW, were near-vertical, and showed at least two episodes of movement in different directions.

Extensive, near horizontal fractures occurred within the rock excavated for the dam and powerhouse. Three open, leached fractures cutting the Georgia abutment strike perpendicular to the dam axis and dip gently toward the river, ending at the rock face between Blocks 7 and 8. Several subhorizontal fractures filled with decomposed rock were exposed during excavation of the spillway blocks. On the South Carolina abutment, several monoliths were excavated below design grade to remove low-angle fractures filled with clay. All of these fractures were subparallel to erosional and top-of-rock surfaces. They were interpreted to be relief joints.

The foundation rock is extensively jointed. Most of these joints are healed with epidote, chlorite, or calcite. Few open joints, except those opened by blasting, were seen below the zone of weathering. However, a few large cavities partially filled with euhedral pyrite, epidote, sintered quartz, and fine transparent needles of an unidentified mineral were seen in the intake and powerhouse areas. Individual joints in the foundation surface are noted on the geologic maps. Where a number of healed joints or veins surface in the foundation they are depicted as chlorite, epidote, or calcite streaks.

e. Bedrock Weathering: Weathering in the Richard B. Russell Project area, as in the rest of the southeastern United States, has been rapid, with thick accumulations of weathered materials between the weathering and erosion surfaces. Faults, shear zones, and open joints allow maximum penetration of meteoric water into rock masses, and mafic dikes provide more easily weathered rock, particularly if they are broken. Since these intersecting, near-vertical (in the dam area) and essentially planar features tend to weather more rapidly than the rock around them, a typical weathering surface in the area altlernates localized pockets of decomposed rock with pinnacles of relatively unweathered rock nearby. This was the type of weathering surface excavated on the South Carolina abutment.

With the exception of a large pocket in the upstream part of Blocks 2 and 3, the Georgia abutment had a relatively regular weathering surface dipping toward the river. This surface extended into Monolith 13, then began to flatten out under the river. The intake and spillway sections, which were mostly excavated into the riverbed, had a thin zone of weathering. In the most extreme case, it was about 10 feet from the top of

rock to fresh, unweathered rock. The tailrace area, also under the river, had a similar zone of weathering. However, in the part of the tailrace immediately behind the pumpback storage units, near-vertical, round-shouldered, open breaks in the rocks caused some problems. These joints were filled with alluvium, including large river-rounded cobbles.

Class I excavation on the excavation contract was greater than estimated (123% overrun). Most of this was removed from Blocks 1-14 and 25-32, with the majority of the overrun in South Carolina. This was, in large part, due to the fact that the exploratory holes in that abutment fortuitously penetrated the pinnacles, rather than the pockets of decomposed rock. The quantity estimated from this information was accordingly low.

f. Leaching and/or Solution Activity: Solution was not a major weathering factor in the metamorphic rocks at the damsite. Formerly calcite-healed veins were open at the surface, where water could dissolve the filling material, but there were no limestones occurring near the surface in the general area. All rock types which were broken to the point that meteoric or goundwater could penetrate them showed leaching. This was usually evident in that feldspars would be a pasty white color, and in some cases would be completely kaolinized. Leaching along joints and faults was of limited extent, being mostly confined to minerals immediately adjacent to the water-conducting break.

Chemical weathering processes such as hydrolysis and oxidation produce from primary minerals materials which can be dissolved or leached as finely divided suspensions or colloids. The silica hardpan, which can occassionally be seen in soil horizons in the general area, is formed from a colloidal product of the hydorlyzation of feldspars. Another product which was present and caused some trouble in excavation for the dam was *kaolinite* clay. This material, removed by groundwater from the rock mass as a finely divided suspension, helped form a slick or "fat" clay which settled in available open joints, including near horizontal relief joints. Since this material lowered frictional resistance to sliding, it was necessary to remove such joints. These "clay seams" were present in both abutments, but were not deep in the Georgia abutment.

A product of oxidation which caused minor trouble was iron salts. Pyrite and other iron sulfides are ubiquitous as accessory minerals in the foundation rocks. As these minerals were oxidized underground, soluble ferrous salts were produced and were leached by groundwater. When this groundwater surfaced in the excavation, the soluble salts were further oxidized into insoluble ferric salts. Some of the intake and spillway blocks were excavated for over a year prior to placement of the last lift of concrete on rock. Where groundwater ran over this rock, ferric salts adhered to the surface. This material proved very difficult to remove, even with a water-blaster.

g. Groundwater: Water table measurements in exploratory core holes prior to excavation showed that the top of groundwater in the right (Georgia) abutment varied from a few feet to 70 feet below top of ground. However, the overall piezometric surface was reported to be very gentle. On the left (South Carolina) abutment, the gradient was much steeper close to the river bluff. However, further from the river it also assumed a very

gentle gradient. This was attributed to possible faulting providing easy drainage near the river. Excavation later proved out this observation.

3.3 Geologic Structure: The geologic structure of the damsite will be discussed under three subtopics: (1) Folds, (2) Joints, and (3) Faults.

3.3.1 Folds: Within the damsite folding on the megascopic scale consists of a northeast striking, steeply dipping to the southeast, isoclinal fold. The dip averages more than 55 degrees. This fold is regional in scale and is likely related to the accretion onto the North American craton of the Avalon terrane.

3.3.2 Jointing: Jointing in the rock at the damsite assumes two main styles. They are: (1) near vertical, intersecting joints, and (2) exfoliation, or relief joints which are sub-parallel to the topographic surface. These two styles will be discussed in turn in the following paragraphs.

The most consistent and persistent joints at the damsite are an intersecting set of near-vertical joints with average attitudes of strike N46°W, dip 89°NE and strike N60°E, dip 89°NE (Design Memorandum 8 - Geology, 1974). This pattern results in the compressive component (Sigma 1) being resolved into a strike of N84°W and the extensional component (Sigma 3) striking N06°E. This stress orientation is consistent with that which the Design Earthquake Report (COE, 1977) determined for the region. Also, the strike of Sigma 3 is consistent with the hypothesis that the Diversion Channel Fault possesses a strike-slip component. Table 2 and Figures 5 thru 8 summarize joint orientations measured during design phase. Locations where measurements were taken are shown on Plate 5.

The majority of the joints within this set have low-asperity faces and are tight. Many have also been healed with calcite, and iron sulfides are also fairly common on the joint faces. The average spacing for these joints is about one foot (30 cm).

The second type of jointing to be discussed is exfoliation or stress relief jointing. These are large-scale structures, sub-parallel to the topographic surface, which were formed by dilation of the rock mass following removal of the overlying confining pressure by erosion. These joints are closer-spaced near the surface than at depth, due to the fact that less residual stress is required to initiate fracturing under conditions of decreased overburden stress. Similar joints were created by excavation for the concrete dam and powerhouse, though on a much smaller scale. Stresses imposed by blasting operations, together with existing stresses, were often of such magnitude as to create joints in some instances, and in others, the combined stresses could only create incipient joints which, when subjected to the added stresses imposed by mechanical excavation equipment during foundation preparation, resulted in a seemingly endless repetition of removing rock from the same area time and again.

3.3.3 Faulting: The major fault within the damsite crosses the Georgia embankment in the vicinity of the Stage I diversion channel and is referred to in this report, and most other C.O.E. reports on this project, as the Diversion Channel Fault. Plates 58 and 60 show the location of the fault

TABLE 2

ORIENTATION OF FRACTURE PLANES

Location*	No. of measurements	Fracture planes Strike	Dip
1,2	301	1. N37W	79NE
		2. N60E	84NW
4	47	1. N43W	77SW
		2. N56E	87SE
5	141	1. N35W	83NE
		2. N83W	82SW
		3. N60E	79NW
6	55	1. N38W	88NE
		2. N62E	88SE
		3. N79W	79SW
7	62	1. N42W	85NE
		2. N80W	74SW
		3. N74E	81NW
8	61	1. N42W	86NE
9	184	1. N37W	86SW
		2. N66W	82SW
		3. N54E	78NW
10	85	1. N38W	88SW
		2. N53E	84NW
		3. N73W	87SW
13	220	1. N34W	79NE
		2. N54E	88SE
		3. N84W	83SW

* Refer to Plate 5 for locations

TABLE 2 (CONTINUED)

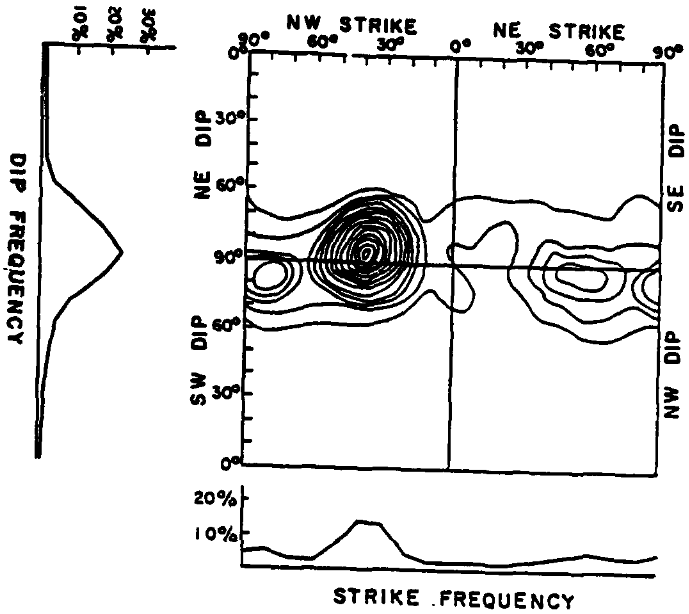
Location	No. of measurements	Fracture planes Strike	Dip
14	234	1. N40W 2. N59E	90 86NW
15	265	1. N46W 2. N60E	89NE 86NW
16	56	1. N45W 2. N60E	86NE 90

* Refer to Plate 5 for Locations

N = 1134
K = 81
N/K = 14
L_K = 10°

FRACTURE PLANES

1. N37W - 84NE
2. N56E - 84NW
3. N80W - 82SW



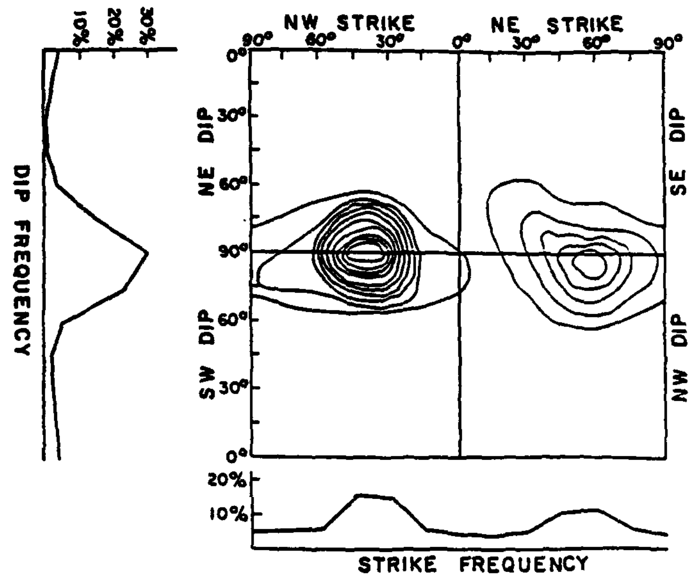
Note: Locations are plotted
on the Geologic Outcrop
Map, Plate 5.

JOINT ORIENTATION AND FREQUENCY DIAGRAMS
LOCATIONS 1, 2, 5, 6, 7, 8, 9, 10 & 13 COMBINED

FIGURE 5

N = 234
K = 36
N/K = 7
L_K = 15°

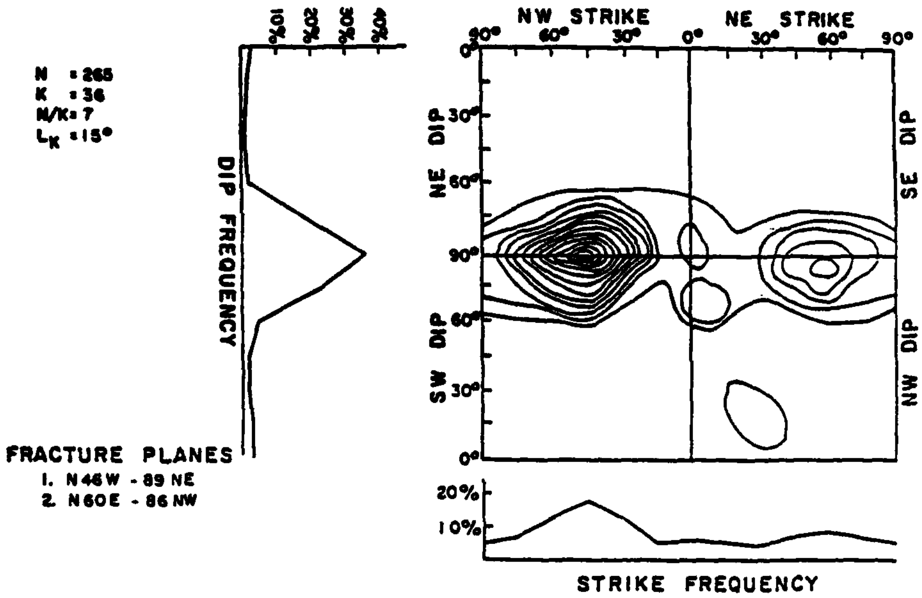
FRACTURE PLANES
1. N40W - 90
2. N59E - 86NW



Note: Locations are plotted
on the Geologic Outcrop
Map, Plate 5.

JOINT ORIENTATION AND FREQUENCY DIAGRAMS
LOCATION 14

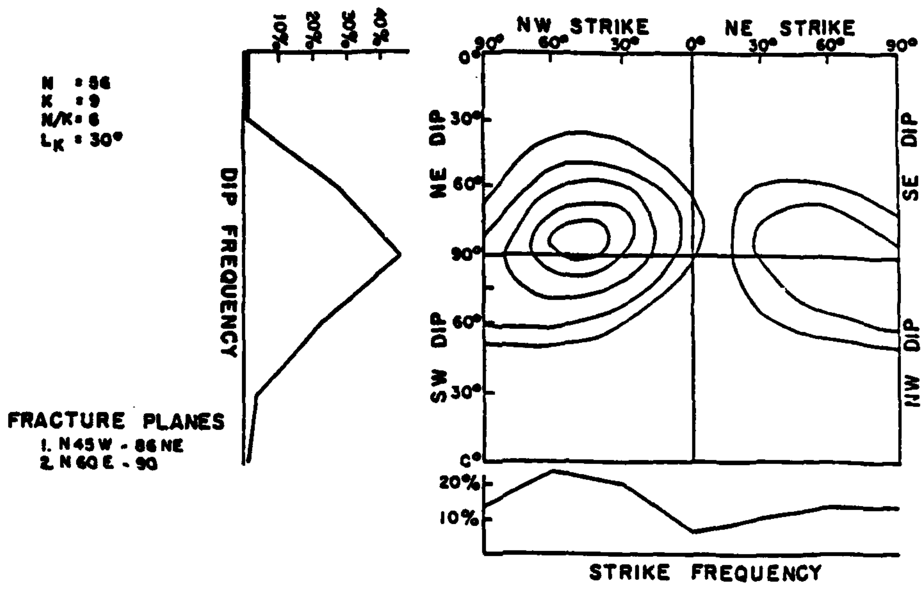
FIGURE 6



Note: Locations are plotted on the Geologic Outcrop Map, Plate 5.

JOINT ORIENTATION AND FREQUENCY DIAGRAMS
LOCATION 15

FIGURE 7



Note: Locations are plotted
on the Geologic Outcrop
Map, Plate 5.

JOINT ORIENTATION AND FREQUENCY DIAGRAMS
LOCATION 16

FIGURE 8

relative to the major features of the project. The existence of this fault was suspected during the exploration phase of the project, but was not confirmed until excavation for the diversion channel exposed it.

The Diversion Channel Fault strikes approximately N10°E and the dip varies from about 45°NW to 55°NW. Indicators of both dip-slip and strike-slip movement are present. These are slickensides, offset lithologic units, truncated lithologic units, and differential weathering across the fault which lead to the hypothesis that the fault has been activated a number of times. The most convincing evidence of this is the presence of a soft gouge zone within the felsite which has intruded into the fault.

The origin of the fault has not been clearly determined, but as Wheeler and Bollinger (1984) point out, "the narrow ends of the Avalon terrane are its only seismically active parts, and are the most likely parts to have had fractures initiated and/or reactivated during the growth, transport, and accretion of the terrane".

Faulting of the Avalon terrane could be the result of reactivated North American cratonic or Iapeton faults. The Diversion Channel Fault may, however, be syngenetic with the Elberton or Danburg batholiths. However, if older than these intrusives, it was almost certainly reactivated by then. Evidence for this line of reasoning is the existence of the felsite filling of the fault and the hydrothermally-altered zone branching off the fault along the upstream limit of the cutoff trench (Plate 77).

3.4 Engineering Characteristics of Overburden Soils:

a. Residual: Residual soils generally blanket the area except in the Savannah Riverbed and where rocks outcrop. The thickness of these soils varies from a few feet to over 50 feet. Residual soils consist of mixtures of sand, silt, and clay classified as SM, SC, ML, CL and CH (Unified Soil Classification System). The soil blanket is generally thin over much of the area. This caused a great deal of difficulty in finding adequate sources of borrow for earthfill.

b. Alluvial: Alluvial soils occur in the Savannah Riverbed as mixtures of sand, silt, and organic material (SP, SM and OL classifications). At the damsite, they are about 25 feet thick. The SP and SM soils are predominant.

A detailed testing program of all major overburden materials is outlined at length in Design Memorandum 17 - Earth Embankments and the Richard B. Russell Earth Embankment Criteria and Performance Report. The testing was done largely in consideration of usage as borrow material, dynamic stability, and in-situ characteristics.

3.5 Engineering Characteristics of the Bedrock Materials: Following is a compilation of information about bedrock material which was taken from Design Memorandum 8 - Geology .

3.5.1 Physical Properties of the Foundation Rock:

a. Prophyritic meta-dacite and associated basic intrusives underly the proposed dam. Field observations and laboratory tests indicate the sound

rock to be strong and competent. Physical tests were made on representative samples of porphyritic meta-dacite and meta-diabase. Cross-hole seismic velocities were measured at various elevations in sound rock under a large portion of the concrete dam foundation, and several up-hole seismic velocity measurements were made for comparison with laboratory sonic velocities. Table 3 is a summary of the physical tests; Table 4 lists cross-hole seismic velocities; Table 5 lists up-hole seismic velocities, and Table 6 lists the laboratory sonic velocities and average Schmidt hammer rebound numbers from intact samples of rock core. Figure 9 is a plot showing the relationship of sonic velocities and Schmidt hammer rebound numbers.

b. Unconfined Compression Tests. An average value of 24,280 psi was obtained from the unconfined compression tests performed on three representative samples of meta-diabase and four representative samples of meta-dacite. The average value of the meta-diabase samples was 20,150 psi and of the meta-dacite samples was 27,390 psi. These values reveal both rock types to be in the high strength classification. An allowable bearing value of 72 tons per square foot will be used in design.

c. Triaxial Tests. Seven samples were subjected to triaxial tests using confining pressures of 1,500 psi and 3,000 psi. The internal friction angles varied from 23 to 58 degrees and the shear stress intercepts (cohesion) varied from 2,000 psi to 8,500 psi (see Table 3). These results further substantiate the high strength of the rock.

d. Shear Strength and Coefficient of Sliding Friction. The direct shear strength values of the samples tested ranged from 2,115 psi to 4,075 psi. Coefficient of sliding friction tests were made on joint surfaces in the rock core. The coefficients of sliding friction varied from 0.98 to 3.61 (see Table 3) and averaged 1.96. An allowable value of 1.0 for the coefficient of sliding friction will be used in design. This value is considered conservative since it is essentially equal to the lowest of seven test values and approximately half the average. The allowable unit shear strength for design purposes is 500 psi. This value was obtained by using the lowest test value and also allowing for low angle joints over 75 percent of an assumed failure plane beneath the surface.

e. Specific Gravity. The average specific gravity was 2.71 for the porphyritic meta-dacite and was 3.00 for the meta-diabase.

f. Modulus of Elasticity. The average value of the static (initial tangent) modulus of elasticity was 7,630,000 psi. The average of three values of the dynamic modulus (Table 3) was 5,700.00. Additional dynamic values were measured, but the results were questionable (see Table 3).

g. Velocity Measurements. Tables 4, 5, and 6 show the results of cross-hole, up-hole, and laboratory sonic velocity measurements. The 44 cross-hole measurements ranged from 12,800 to 16,000 feet per second. The laboratory sonic velocity measurements ranged from 12,000 to 19,600 feet per second. All measurements were of sound rock velocities or between sound rock zones in adjacent drill holes. Figure 9 shows the excellent correlation between the average Schmidt hammer rebound numbers and sonic velocities. The sonic velocity measurements were all made on dry samples

TABLE 3

SUMMARY OF PHYSICAL TESTS

Lab no. rock type	Hole no. sample depth	Bulk specific gravity	Unconfined compressive strength PSI	Poisson's ratio	Normal load PSF	Modulus of elasticity PSI x 10^6		Direct shear strength-PSI		Coefficient of sliding friction (sliding load/normal load)		Angle of internal friction degrees	Cohesion PSI
						Static	Dynamic	Rock	Concrete on rock	Rock on rock	Concrete on rock		
IM 2512 meta-diabase	C-146 21'	2.94	23,550	0.29	5,000	7.14	0.54*	3,590	440	1.67	1.27	36	4,600
IM 2513 qtz-feld-gneiss	C-130 28'	2.70	26,720	0.24	10,000	5.20	5.78	3,250	200	1.51	1.15	23	8,300
IM 2514 meta-diabase	C-7 35'	3.04	10,960	0.21	13,000	12.90	0.40*	4,075	152	1.49	----	58	2,000
IM 2515 qtz-feld-gneiss	C-129 15'	2.70	31,940	0.30	4,000	5.20	7.43	2,180	165	3.61	----	26	8,500
IM 2516 meta-diabase	C-29 38'	3.02	25,930	0.25	9,000	13.79	1.05*	3,190	90	1.37	----	40	5,000
IM 2517 qtz-feld-gneiss	C-132 30'	2.70	34,350	0.25	12,000	3.18	3.90	2,390	265	3.08	----	41	7,300
IM 2518 qtz-feld-gneiss	C-144 38'	2.72	16,540	0.21	20,000	5.97	0.24*	2,115	134	0.98	1.08	**	**

Remarks:
*Sample contained healed fractures which probably interferred with frequency readings.
**Indeterminate, probably due to the variable strength of the steeply dipping fractures present in the rock.

TABLE 4

CROSS-HOLE SEISMIC VELOCITIES

Shot location hole no.	Shot elevation	Hydrophone location hole no.	Hydrophone elevation	Velocity fps
C-127	327.0	C-148	327.0	11,300
C-127	327.0	C-146	327.0	13,400
C-127	327.0	C-147	327.0	12,800
C-127	327.0	C-145	327.0	14,500
C-127	315.0	C-148	315.0	14,200
C-127	315.0	C-146	315.0	13,400
C-127	315.0	C-147	315.0	17,100
C-127	315.0	C-145	315.0	17,000
C-130	300.0	C-129	300.0	15,000
C-130	300.0	C-131	300.0	13,800
C-130	290.0	C-129	290.0	17,600
C-130	290.0	C-131	290.0	16,900
C-131	300.0	C-129	300.0	16,700
C-131	300.0	C-132	300.0	12,200
C-131	290.0	C-129	290.0	18,100
C-131	290.0	C-132	290.0	16,800
C-130	303.5	C-151	289.5	11,000
C-130	303.5	C-152	285.0	11,500
C-130	303.5	C-151	299.5	10,500
C-130	303.5	C-152	295.0	11,200
C-152	285.0	C-151	285.0	10,200
C-152	285.0	C-153	285.0	9,500
C-152	295.0	C-151	295.0	10,000
C-152	295.0	C-153	295.0	9,800
C-152	288.5	C-151	288.5	10,000
C-152	288.5	C-153	288.5	10,000
C-134	293.4	C-164	290.4	10,000
C-134	293.4	C-135	290.4	9,500
C-134	293.4	C-164	280.4	16,500
C-134	293.4	C-135	280.4	14,800
C-136	286.2	C-137	286.2	10,700
C-136	286.2	C-135	286.2	10,100
C-136	276.2	C-137	276.2	10,400
C-136	276.2	C-135	276.2	9,800
C-155	278.2	C-137	278.2	11,300
C-155	278.2	C-138	278.2	12,700
C-155	288.2	C-137	288.2	11,500
C-155	288.2	C-138	288.2	12,500
C-164	275.0	C-153	275.0	13,900
C-164	250.0	C-153	250.0	18,400
C-154	284.1	C-164	284.1	14,400
C-154	284.1	C-153	284.1	14,300
C-154	294.1	C-164	294.1	9,200
C-154	294.1	C-153	294.1	9,100

TABLE 5

UP-HOLE SEISMIC VELOCITIES

Hole no.	Shot elevation	Hydrophone elevation	Velocity
C-153	250.0	300.0	12,800
C-164	240.0	290.0	16,100
C-152	247.0	297.0	14,700
C-151	240.0	290.0	15,600

TABLE 6

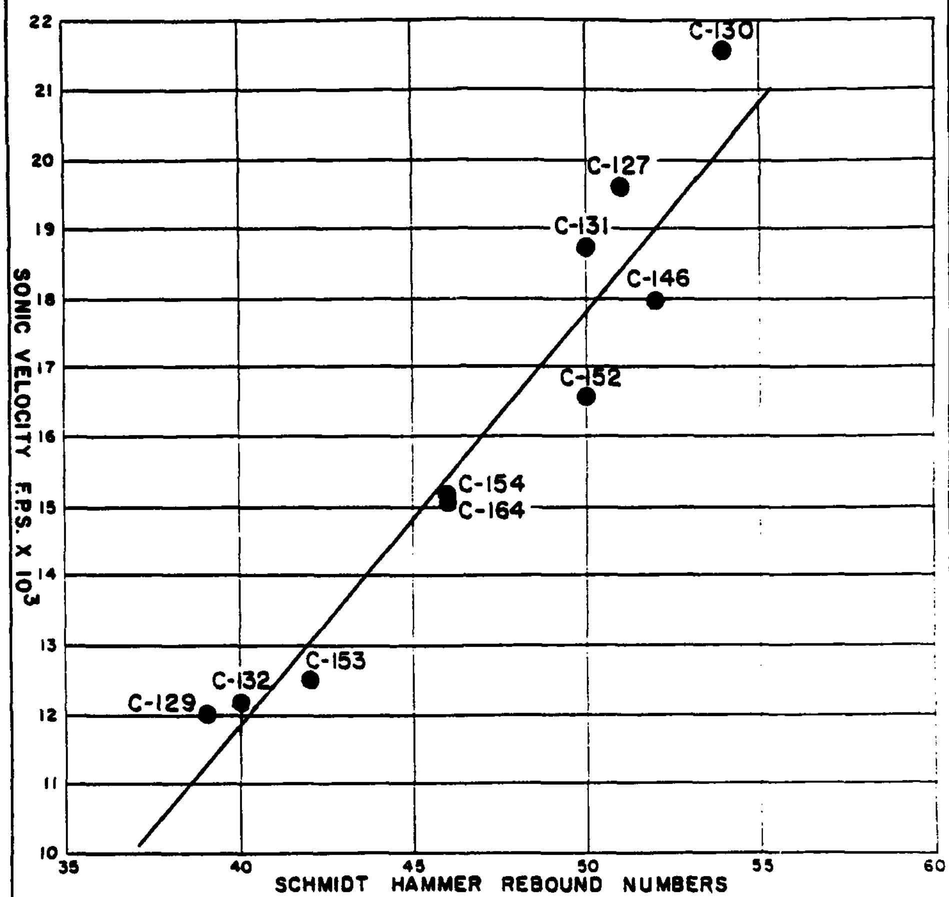
SONIC VELOCITIES AND AVERAGE SCHMIDT REBOUND NUMBERS

Hole no.	Elev.	Rock type	Rebound no.	Velocity
C-127	315	Meta-diabase	51	19,600
C-129	292	Qtz-feld-Gneiss	39	12,000
C-130	292	Meta-diabase	54	21,600
C-146	314	Meta-diabase	52	18,000
C-131	290	Qtz-feld-Gneiss Flaser Texture	50	18,800
C-132	292	Qtz-feld-Gneiss	40	12,100
C-152	285	Qtz-feld-Gneiss Flaser Texture	50	16,600
C-153	286	Qtz-feld-Gneiss	42	12,500
C-154	280	Qtz-feld-Gneiss	46	15,200
C-164	271	Qtz-feld-Gneiss Flaser Texture	46	15,100

TABLE 7

CORRELATION OF RQD AND SEISMIC VELOCITIES

Description of rock quality	RQD percent	Seismic velocities (fps)
Very poor	0-25	6,000 - 8,500
Poor	25-50	8,500 - 11,000
Fair	50-75	11,000 - 13,500
Good	75-90	13,500 - 15,000
Excellent	90-100	15,000 -



RELATIONSHIP OF SONIC VELOCITIES AND SCHMIDT HAMMER REBOUND NUMBERS

FIGURE 9

with no pressure applied. Tables 4-6 tabulate the lowest elevation cross-hole velocity, up-hole velocity, and sonic velocity measurements and elevations. The following conclusions are the result of velocity studies to date at the Richard B. Russell project:

(1) Velocities are greater in the meta-diabase (specific gravity 3.0) than in the meta-dacite (specific gravity 2.71). Laboratory sonic velocities of three meta-diabase samples varied from 18,000 to 21,600 feet per second. Of seven meta-dacite samples, only one had a sonic velocity above 17,000 feet per second, and the average was 14,600 feet per second.

(2) Alteration of feldspars to kaolinite (less than 10 percent kaolinite) apparently causes a substantial reduction in sonic velocities, but not in unconfined compressive strengths. Evidence for this statement is based on tests performed on the rock from hole No. C-129. Petrographic analysis, physical tests, and sonic velocity measurements were all performed on samples from this hole. The kaolinite content was approximately 8 percent and the sonic velocity 12,000 feet per second (the lowest value of 8 samples); however, the unconfined compressive strength was 31,940 psi (the second highest of seven tests).

(3) For velocity ratio studies, sonic velocities probably should be measured under saturated conditions using 3,000 to 5,000 psi uniaxial applied stress. Laboratory velocity (VL) measurements on sound intact samples of rock normally give higher velocities than do field seismic (VF) measurements). The velocity ratio, VF/VL, has been proposed as a method of evaluating the degree of jointing and fracturing in the rock. Theoretically, the difference between the two values is caused by discontinuities in the rock. In a massive, tight, fresh rock mass, the velocity ratio should approach unity. As discontinuities increase, the velocity ratio should get smaller. At this site, the theory appears to be valid in the meta-diabase rock, but in the more predominant meta-dacite rock, it does not hold up. In almost every case, the laboratory sonic velocity of intact specimens of meta-dacite was less than the field seismic velocity in the same boring. This would give velocity ratios greater than unity. Up-hole seismic velocities were measured in the field to see if direction of measurement was causing this discrepancy. The results did not indicate this to be the answer. It was finally concluded that the conditions of in-situ stress and saturation of the rock in the ground must be at least a partial explanation of the failure of the velocity ratio method in this case.

(4) Additional subsurface investigations must be made to determine why the cross-hole velocities were so slow (10,000 fps) between holes C-135, C-136 and C-137, C-151 and C-152, and C-152 and C-153. Deep weathering or faulting between these holes could explain the abnormally low velocities.

(5) Velocities increase with depth.

(6) Comparison of seismic velocities with the rock quality designator (RQD) is more meaningful at this site than a comparison of the velocity ratio with the RQD. Table 7, on page 30 shows approximate, assumed correlations between seismic velocities (including refraction, cross-hole, and up-hole) and the RQD for the Richard B. Russell area. Enough

measurements have been made to indicate a good degree of confidence in the correlations.

4.0-CD SPECIAL DESIGN CONSIDERATIONS - CONCRETE DAM

4.1-CD Uplift Forces: Uplift on the base of the concrete dam was assumed to vary according to the following criteria: full tailwater at the toe of the dam to tailwater plus one-half the difference between headwater and tailwater at the heel of the dam. Uplift pressure is considered to be effective over the entire base of the dam. When examining the stresses at any horizontal plane through the structure above the base, the uplift pressure is assumed to be 50 percent effective over the entire area of the plane under consideration.

To contend with uplift, a system of gallery drains was drilled into the foundation rock. Angled 15 degrees in the downstream direction, the drain holes were drilled to an average depth of 80 feet into the foundation rock beneath the dam. These holes were cased within the concrete structure, allowing several inches of "stick-up" above the gutter elevation. The tops of each drain allow water to flow out of the drains and into the gallery gutter, wherein it is led to one of two sump pumps for final elimination from the inspection gallery. (See gallery drain details on Plate 50.)

In order to effectively measure uplift pressure, a pattern of uplift cells was designed and installed for measuring and reading pressures. The cells permit water to enter through well screens and act against pressure gauges. The pressure gauges are located conveniently in the inspection gallery where they may be read individually by monoliths. For details of uplift cells, refer to Plates 53, 54, and 57.

4.2-CD Foundation Bearing: In design, an allowable bearing of 72 tons per square foot (or 1000 psi) was established. In the testing of the seven rock samples previously mentioned under Section 3.5.1 average strengths of 27,390 psi for meta-dacite and 20,150 psi for meta-diabase were determined. The average static modulus of elasticity for all samples was 7,630,000 psi. The lowest unconfined compression strength result of 10,960 psi still yields a safety factor of 10.96 or nearly 11.

4.0-EE SPECIAL DESIGN CONSIDERATIONS - EARTH EMBANKMENTS

4.1-EE Overlap Grout Holes: In order to obtain effective grout coverage in the regions of the Georgia earth embankment where it abuts the concrete dam structure, a series of angled fan grout holes was designed. Drilling and grouting this pattern of grout holes permitted a consolidation of the embankment grout curtain and the grout curtain beneath the concrete dam. This fan arrangement is best understood by referencing Plates 86 thru 92.

4.2-EE Fault Treatment: The fault uncovered during diversion channel excavation proved to be an item for modest concern during embankment design. Thought to be, for all purposes relatively inactive, its large size dictated some special design consideration. (Water losses during pressure testing had been large in the vicinity of the fault also). A blanket or consolidation type grout program was designed for intercepting the fault plane at various elevations determined by the plane of its orientation. In addition, a surface grout cap was included in the contract drawings for adequately covering the excavated surface along the grout curtain from which

the consolidation holes would be drilled and grouted. Details of the consolidation grouting, as well as the concrete cap, can be found on Plates 82, 108, and 109. The results of the consolidation grouting program, which was field-modified somewhat, are discussed in Section 7.1.4-EE.

4.0 PH SPECIAL DESIGN CONSIDERATIONS - POWERHOUSE

4.1-PH Hydrostatic Uplift: The powerhouse structure was designed to utilize gravity design, an extensive rock bolt network, and foundation relief wells to counteract hydrostatic uplift forces. Section 8.0-PH describes the rock bolts and anchors utilized in the structure, the quantity and locations, and the engineering rationale for their employment. Section 10.0-PH covers the relief well system utilized. Design studies concluded that the combination of elements which were used would be preferable to a totally gravity design. This conclusion was based upon both economic and constructabilty considerations.

4.2-PH Spillway Training Wall: The spillway training wall, which extends downstream from the South Carolina side of Unit 8, not only has to carry the vertical loading imposed by the observation tower, but also has to resist rotational and overturning forces imposed by tailrace and spillway water currents. To meet the stability criteria, a series of twelve (12) twenty-three strand, high-capacity Type "C" rock anchors (Plates 127 and 129) and a series of sub-horizontal Type "A" rock bolts (Plate 128) were incorporated into the design. In addition to this, the designers requested that the wall be "keyed" into the rock and the Area Office staff added five (5) twenty-foot long Type "D" rock bolts which were installed vertically through the second lift of concrete and grouted into the rock with polyester resin. After the grout set up, the locking nuts on the bolts were torqued with a manually operated wrench rather than stressing with a hydraulic jack as called for by the specifications. These Type "D" bolts were used primarily to maintain concrete-to-rock contact and to guard against the possibility of Unit 8 discharge water exerting any uplift forces against the bottom of the wall.

5.0-CD EXCAVATION PROCEDURES FOR COMPONENT PARTS - CONCRETE DAM

5.1-CD Excavation Requirements: The excavation phase of dam and powerhouse construction was undertaken as a separate contract to expedite completion of the project. The point of separation was that the excavation contractor would take the excavation to design grade, clean up by mechanical means only (with some washing of the surface provided for in the contract), perform needed dental excavation, drill exploratory holes as needed to confirm the suitability of the foundation rock, then demobilize. The concrete dam contractor would perform foundation preparation by removing a minor amount of rock by hand methods, wash the entire foundation, then construct the dam on the resulting foundation.

The excavation contract defined two classes of excavation. Class I was that material which could be removed by diligent use of a single shank hydraulic ripper on a D8H bulldozer. Class II was that material which had to be systematically drilled and blasted. Within Class II, "sound rock" was further identified as rock that is hard, intact, fresh, and essentially unweathered.

Slopes were set based on the material being excavated. Class I excavation slopes were variable, but in no case were they allowed to be steeper than 1V on 1.5H. Firm rock (Class II rock not classified as sound) slopes were to be 1 on 1. Sound rock slopes were vertical. A bench or berm not less than 6 feet wide was to be left between firm and sound rock slopes.

5.2-CD Excavation Grades: Excavation grades as designed and directed in the field are depicted on Plate 16. As-built foundation grades after foundation rock removal and preparation are depicted as topographic surfaces on Plates 18 through 49.

Directed changes occurred in 3 main areas. The first of these was the Georgia abutment. After Class II excavation reached design grade in Monoliths 2 and 3, a large pocket of decomposed material appeared to be both weathered and hydrothermally altered. It was easily removed by a Caterpillar 245 backhoe.

The second area is the spillway foundation. In this area (probable) relief joints in a near horizontal attitude occurred below design grade. The first of these was discovered when communication was noted between holes RBR10A and RBR18 in Monolith 19. Material filling the seam was decomposed and crushed rock fragments of sand and gravel size. Both metadacite and metadiabase were fragmented where intersected by the seam, and lateral displacement of near-vertical dikes along the seam was 0 to 6 inches.

The third area was in the South Carolina abutment. Monoliths 26, 27, and 28 were founded at lower elevations to remove clay seams below design grade. These seams were also near horizontal and were probably relief joints. Exploration drill holes indicated that these seams probably terminated at their intersection with metadiabase dikes. Monoliths 27 and 28 were lowered by modification after the excavation contractor had essentailly completed his contract work.

Minor excavation had to be done on the concrete dam contract. The dewatering sump in the Block 9 inspection gallery was designed to be founded below the excavation surface left by the Lane Company. In order to avoid interference between contractors, the Dravo-Groves contract was modified to allow them to blast and remove the unwanted rock.

5.3-CD Dewatering Provisions: When excavation was begun in 1977, the construction area lay between two cofferdikes approximately 1000 feet upstream and downstream of the concrete dam. A sedimentation dike, constructed of sand and gravel, created a triangular pond immediately downstream of the downstream cofferdike. The water level in this pond responded to water level changes in Clarks Hill Lake. The spillway and part of the intake areas were low, swampy, wandering watercourses immediately after unwatering.

The excavation contractor dug a downstream sump and then built an elevated (on random fill) road immediately upstream of the construction area. Water which ponded upstream of this road was led across the Monolith 22 location by a ditch with piled fill on either side to the downstream sump. From the downstream sump, the water was pumped over the downstream cofferdike into the sedimentation pond. Since the sedimentation dike was permeable to water, but not to silt and clay, the water filtered into Clarks Hill Lake, leaving the sediment behind during periods in which Clarks Hill Lake was low. When the lake was high, water filtered in reverse flow through the dike cleaning the accumulated silt and clay from within the sand and gravel fill.

When excavation was proceeding at its fastest pace in the tailrace and powerhouse areas, two 8-inch pumps were required to empty the excavation into the downstream sump. During this period, work on the concrete dam was going on, adding more water to the amount of dewatering necessary. A large pump of unknown capacity would then move this water from the sump into the sedimentation pond. This plant was capable of keeping up with runoff and groundwater flows in all but the heaviest thunderstorms. Exact quantities of water pumped were not kept, since the contractor was paid by the month instead of by a unit quantity. Appendix B was compiled from contractor records available for a 6-month period in 1978.

5.4-CD Overburden Excavation: Overburden excavation began with the dredging of river sand prior to dewatering. The dredged sand was moved as a slurry to the sand stockpile by pipeline with the help of two booster pumps. When dredged material was considered by Corps inspectors to be sufficiently contaminated by muck, it was sent to the waste pile. By contract, the riverbed was dredged to within 2 feet of rock.

After dewatering, Class I excavation was accomplished with bulldozers (Caterpillar D8's, end loaders (Caterpillar 988's), and off-road dumps (Caterpillar 769's). The dozers broke up and piled the material, the loaders filled the dumps with it, and the dumps hauled it to the stockpile, or more commonly to the waste pile. On the abutments, the Class I excavation material was pushed downhill or over the bluff where it was loaded. There was a contract overrun of 123% in Class I excavation in the concrete dam and powerhouse areas.

5.5-CD Rock Excavation: The contractor began drilling and blasting operations in November 1977, on the South Carolina abutment. He began drilling on the Georgia side of the excavation in January 1978. After that time, he always worked at least two areas at the same time. At the beginning of operations, the area from the downstream portion of Monoliths 8 through 16 was held up by the Corps (pending completion of design for the powerhouse) to allow no excavation below 277 MSL. This prohibition was relaxed in April 1978.

The contractor drilled his blast holes with air-trac drills, working as many as six at one time. Although the contract specifications allowed holes up to 6 inches in diameter, the contractor elected to use 3 and 3-1/2-inch holes on his production blasts. Blasts to grade were limited by contract to 2-1/2-inch diameter holes. Maximum lifts as shot were 16 to 20 feet. Although a maximum lift of 5 feet for shots to grade was specified, when there were only 7 or 8 feet of rock to be excavated below ripper refusal this was shot in one lift. The most common pattern used in production blasts was a 5-foot by 6-foot rectangular pattern. This pattern was closed up for blasts to grade.

The production blasts, as designed, generally achieved good breakage, but backbreak was intensified by the jointed nature of the metamorphic rock. Healed and incipient joints were opened by high pressure gasses. Calcite sheets could often be easily lifted from formerly healed joints now opened by blasting in the deeper powerhouse shots. It was difficult to hold backbreak to less than 6 or 7 feet in production blasts. This tendency of the rock to split along pre-existing joints particularly affected the slopes in the downstream portions of Blocks 6 through 16. After foundation preparation "stairstep" slopes were common in these blocks.

Another difficulty encountered in drilling and blasting operations was the problem of caving holes. As previously stated, the weathering surface in the South Carolina area was one of pinnacles and pockets. Where hard rock surrounded and protected pockets or crevices filled with decomposed rock materials, penetrating blast holes often caved repeatedly. These pockets and crevices affected blasts for 2 or 3 consecutive lifts in some cases.

The Lane Company used DuPont explosives in production blasts. Holes were loaded with ANFO prills and detonated by electric caps in HDP primers where possible. Where water in the holes or oversized holes dictated, Tovex 700 cartridges of 2 and 2-1/2-inch diameter initiated with EM caps in Detaprime UA primers were used. Blasts were usually delayed in a chevron pattern to give a humped muck pile which facilitated loading with a front end loader. In the harder rock deep in the powerhouse excavation, a rectangular shot would sometimes be delayed in rows toward one corner (where two free faces existed) to get better breakage. Powder factors were usually one pound per cubic yard or less in firm rock. These ranged up to two or two-1/2 in the powerhouse trench. Blast No. BP1205, a sinking shot in the powerhouse area with holes angled toward and delayed to the center of the shot, had a factor of three.

Shot rock was loaded onto Caterpillar 769 off-road dumps by Caterpillar 988 loaders assisted by D8 bulldozers. As the rock was loaded, Corps

inspectors designated it as firm or select rock. The rock was then taken to the appropriate firm rock or select rock stockpile. All the rock removed from the dam and powerhouse excavation was used in construction. Firm rock was primarily used to build construction roads. Select rock was used as filter material and rock fill in the earth embankments.

5.6-CD Line Drilling and Presplitting: The excavation contract specified line drilling (spacing on holes not to exceed twice the diameter) "whenever a required vertical surface is less than four feet in height (depth); and for a horizontal distance of five feet on either side of all external corners for the full depth of the vertical surfaces." Presplit blasting was required on all surfaces inclined 1V on 1H or steeper which were 4 feet or greater in height and which were not required to be line drilled. Spacing on presplit holes was to be 20 to 36 inches.

Presplit holes were loaded with Hercules Powder Co. Hercosplit WR. Most blasts were loaded solid with these 2-foot long (.6 lb/foot) cartridges, but in some cases, the holes were deck loaded with cartridges spread a foot or more apart. The Hercosplit is a nitroglycerin-based explosive and was initiated with primacord. Tovex 1, a water gel explosive in 50-foot long, 1-inch diameter, plastic tubes with internal primacord was tried on the South Carolina abutment, but the Contractor discontinued its use after one blast.

5.0-EE EARTH EMBANKMENT - EXCAVATION

5.1-EE Excavation Grades: The excavation grades for the embankments are shown on Plates 60, 61, and 62. These plates show both the design grades and the "as-built" grades of the cutoff trenches. For more details, see the Embankment Performance and Criteria Report for this project.

5.2-EE Dewatering: Contract provisions for dewatering of the embankment considered only the diversion channel between the closure dikes. Design grades for the cutoff trenches assumed that "slope to drain" configuration could be achieved and detailed dewatering provisions were not written into the contract. This turned out not to be the case for all sections of the cutoff trenches and this condition required pumping and/or the blasting of drainage trenches through high points of the foundation rock. The area between Station 5+50 and Station 6+50 (referred to as the "bathtub" and whose treatment is presented in detail in Section 7.3-EE) presented the greatest problems for dewatering.

Dewatering of the diversion channel between the closure dikes required pumping from the time of initial closure until fill placement began (a period of eight months duration). The dewatering system consisted of one eight-inch submersible pump which proved to be effective except on two occasions when excessive seepage through the upstream closure dike flooded the cutoff trench. Details concerning the closure dike seepage are presented in the Embankment Performance Report.

5.3-EE Overburden Excavation: Excavation of overburden for the cutoff trenches included alluvium, residual soils, and intensely weathered rock. Overburden excavation was classified as that material which could be removed by equipment such as scrapers, dozers, rippers, and hydraulic excavators

without resorting to drilling and blasting and was paid for under Bid Item 20: Excavation, Common. Ripper refusal criteria was based upon the use of a single-shank, parallelogram, hydraulic ripper mounted on a Caterpillar tractor Model D8H, 46A (or equivalent). All cutoff trenches were excavated to ripper refusal (as a minimum) except in the transition slopes, the "bathtub", and the diversion channel crossing. All material which was excavated was classified by a Government Inspector and was either stockpiled for future use or was wasted, based upon their classification.

The side slopes of the excavation were sloped to a 1V on 1.5H and presented no stability problems. The contractor was required, however, to maintain the slopes by constantly monitoring and scaling, and to provide whatever support might be required.

5.4-EE Rock Excavation: The earth embankments cutoff trench was excavated to ripper refusal using a Caterpillar D8H 48A, equipped with a single-shank parallelogram ripper. Excavation below this level was performed only in isolated instances for the following purposes: (1) to remove high "knobs" of hard rock; (2) to provide drainage; and (3) to provide a smooth transition into the diversion channel. All rock excavation for the preceding purposes was performed using explosives. A total of approximately 47,500 CY of rock was excavated under the embankments contract, with the vast majority of this quantity being required to achieve a smooth transition into and across the diversion channel (Plate 82). The only other area with a substantial quantity of rock excavation was in the South Carolina abutment, where approximately 7,500 CY of rock was blasted to achieve a smooth transition. All other blasting consisted of small pop shots to remove small knobs or ledges of rock to achieve drainage or to provide for more efficient construction of the embankment.

Blast patterns varied from 4x4' to 5x7' to individual holes. The powder factor was on the order of 1.1 lb/yd and gelatin dynamite was the most commonly used explosive. All shots were detonated with millisecond delay electric blasting caps. Subdrilling was normally held to a maximum of 1 foot with a minimum of 3 feet of stemming. Lightning detectors and seismographs were required for all blasting operations.

5.0-PH EXCAVATION PROCEDURES FOR COMPONENT PARTS - POWERHOUSE

5.1-PH General: Excavation for the powerhouse structure was performed under a separate contract by the Lane Corporation of Meridian, Connecticut. Procedures, methods, and equipment are presented in detail in Section 5.0-CD. The only excavation performed under the Powerhouse Contract (other than foundation preparation and switchyard excavation) was that required for sumps in the erection bay and areas which were not shot to grade under the Excavation Contract.

5.2-PH Erection Bay Sumps: In the erection bay area, three (3) sumps had to be excavated in the foundation rock. This work was accomplished through the use of jack hammers, prybars, and Bristar. Bristar is a commercially available demolition agent which is non-explosive and which fractures the rock because of its expansive properties.

5.3-PH Penstock Drain Line: The penstock drain line for Service Bay 2 required blasting to remove high areas of rock. This blasting was performed by Piedmont Explosives, Inc. with vibration monitoring and evaluation by Vibra-Tech South. A total of five (5) shots were made with maximum recorded particle velocity being 0.91 ips; acceleration being 1.48 g's with a particle displacement of 0.002 inch. Maximum pounds of explosive was 7-1/2 lbs. with a maximum 1 lbs/delay = 1/2 lb. See Table 8 for tabulated blast data.

5.4-PH Vertical Rock Face: The vertical rock face between Service Bays 1 and 0 required rock removal to allow for the minimum required concrete thickness in the vertical wall. This wedge of rock was line drilled with hand-held air-powered rock drills and removed using Bristar.

TABLE 8

BLAST DATA - POWERHOUSE EXCAVATION

Date	1-19-82 (#1)	1-19-82 (#2)	1-19-82 (#3)
Number Holes	28	37	24
Diameter (inches)	1-1/2	1-1/2	2-1/2
Depth (inches)	9"-14"	15"-18"	15"-18"
Spacing (feet)	18"	2'	2'
Burden (feet)	24"	2'	2'
Explosives Weight (lbs)			
40% Gel	5-1/3	7	4-1/2
Total	5-1/3	7	4-1/2
Atlas Rockmaster Delays	1,3-6,8-16	1,3-6,8-21	1,2-6,8-16
Maximum Holes/Delay	2	2	2
Maximum Lbs./Delay	6 oz.	6 oz.	6 oz.
Stemming (inches)	6"-10"	3"-15"	3"-15"
Theoretical Delay	25 ms	25 ms	25 ms
Duration of Blast	550 ms	750 ms	550 ms

TABLE 8 (CONTINUED)

BLAST DATA - POWERHOUSE EXCAVATION

Date	1-20-82 (#4)	1-20-82 (#5)
Number Holes	20	12
Diameter (inches)	1-1/2	1-1/2
Depth (inches)	24"-9"	16"
Spacing (feet)	2	2
Burden (feet)	2	2
Explosives Weight (lbs)		
40% Gel	7-1/2	2-1/3
Total	7-1/2	2-1/3
Atlas Rockmaster Delay	1-5,7-19,21,25	7-13
Maximum Holes/Dalay	1	2
Maximum Lbs./Delay	8 oz.	6 oz.
Stemming (inches)	1.5"-6"	13"
Theoretical Delay	25 ms	25 ms
Duration of Blast	975 ms	250 ms

6.0-CD FOUNDATION EXPLORATIONS DURING CONSTRUCTION - CONCRETE DAM

The excavation contract included an item for four-inch rock core "to determine the condition of the rock prior to and during excavation". In the fall 1977, 14 borings were drilled to explore unusual conditions revealed by clearing operations. These were limited primarily to near vertical fault and shear zones.

During early excavation on the South Carolina abutment, a number of "clay seams" had been encountered. Since some of these seams had been quite extensive, it was necessary to determine their existence and areal extent below planned foundation grade. Although they were difficult to pick up in core borings, a series of holes was laid out in both abutments and the spillway area, where deep horizontal joints would not be intersected by the excavation. Eventually, 74 borings were done during this program.

Four-inch borings done in the west abutment area encountered clean, leached joints which dipped at a low angle below design grade. To explore these joints extensively and economically, the excavation contract was modified to allow air-trac drilling without water. The drills were observed by Resident Office personnel and depths were recorded for color changes in cuttings, loss of cuttings, and water or mud production. The recorded depths and types of phenomena were then correlated and three-point problems worked out to define the attitude of the relief joints. The holes were drilled on 20-foot centers in Monoliths 1 and 4. There were approximately three joints striking across the dam axis and dipping in an easterly direction parallel to the axis. These joints were exposed when the 35-foot "step-up" face between Blocks 7 and 8 was revealed by excavation.

No exploratory investigations were done on the concrete dam contract.

6.0-EE FOUNDATION EXPLORATIONS DURING CONSTRUCTION - EARTH EMBANKMENTS

The initial contract estimate for exploratory borings called for 2,300 lineal feet of NX size coring. Field conditions indicated the need for additional coring and a modification for 3,100 lineal feet of additional drilling was issued. Of the total contract amount of 5,400 lineal feet of exploratory drilling, a total of 4,629.2 feet were actually drilled. The locations of some of the exploration drill holes are shown on Plates 67 and 68.

Logs of a limited number of the exploratory borings made by the Contractor are included as Appendix J. The contract specifically required the Contractor to provide complete drill logs for each boring made, and went into great detail regarding the educational and experience requirements for the Contractor Quality Control organization's resident Geologist, as well as listing all items which were to be included on the drill logs and on the core boxes. For reasons unknown these requirements were not met, and hence, much of the core recovered from the exploratory borings is of extremely limited value.

6.0-PH FOUNDATION EXPLORATIONS DURING CONSTRUCTION - POWERHOUSE

During construction of the Richard B. Russell powerhouse, foundation investigations consisted entirely of personal inspection and evaluation of the foundation by the Project Geologists. The entire foundation area was mapped geologically and personal inspection and approval by one of the Project Geologists was required prior to concrete placement. In addition, drilling operations for rock bolts, relief wells, and grout holes were monitored periodically and driller's reports, drill cuttings, hydraulic pressure test results, and grout operations were evaluated by the Project Geologists. No borings or tests were made for the express purpose of investigating foundation conditions during construction of the powerhouse.

7.0-CD UNUSUAL OR UNANTICIPATED CONDITIONS ENCOUNTERED DURING CONSTRUCTION - CONCRETE DAM

Several geologic conditions encountered in the concrete dam foundation caused problems in excavation and foundation preparation. However, these conditions were not totally unexpected. The extent or location of these features was not predicted in some cases, but their presence or the presence of similar features had been noted by SASEN-GG. These conditions will be covered in Section 9.0-CD.

One geological feature which seemed unusual to the resident staff was the presence of areas in the foundation which had been subjected to rotational stress, probably in a near horizontal plane. Evidence of this can be seen in the distorted metadiabase dikes and in tension cracks or tears within the metadacite country rock. Such evidence was found in Monoliths 13, 15, 16, 24, 25, 27, and 28, but it is most convincing in Monolith 21. There it appears that the semiplastic rock (probably during a metamorphic phase) was twisted to bend the previously intruded dikes. At sites of most extreme dislocation small tears, sometimes an echelon, were produced. Also found within the disturbed area were blebs of "bull" quartz and pockets of euhedral, pink calcite.

7.0-EE UNUSUAL OR UNANTICIPATED GEOLOGIC CONDITIONS ENCOUNTERED DURING CONSTRUCTION - EARTH EMBANKMENTS

Considering the sheer size of the Richard B. Russell embankments and the geologic terrane within which the project is located, suprisingly few unusual or unanticipated geologic features were encountered during construction. The specialized treatment of selected areas due to adverse geologic conditions is described in the following paragraphs by type of treatment.

7.1.EE <u>Diversion Channel and Fault Zone Treatment</u>: A major fault zone bisects the Georgia embankment in the vicinity of the old diversion channel. This fault trends approximately N10°E and dips about 50 degrees to the northwest. Movement along the fault appears to consist of both a right lateral strike component and a normal dip component. The fault itself has been intruded by a felsic dike shortly after faulting. Appendix A contains the complete petrographic and stress history analyses performed by the South Atlantic Division Laboratory.

Although the rock types on either side of the fault were identical (i.e., quartz-feldspar gneiss cut by mafic dikes), the appearance and geotechnical properties were different. The weathering profile was much more pronounced and extended to much greater depths in the hanging wall than in the footwall. The magnitude of this difference was clearly illustrated during excavation of the fault for dental treatment. Excavation was performed with a Cat 245 backhoe which experienced little or no difficulty in excavating the hanging wall; however, excavation of the footwall could be performed only with extreme difficulty, if at all.

In addition to the fault zone itself, the embankment crossing of the diversion channel was of great concern to the designers as well as field engineers. This was the highest section of the embankment and had abrupt

foundation elevation changes on either side. For these reasons, the most thorough, conservative treatment was performed in this area.

7.1.1-EE Excavation: The steep side slopes of the diversion channel were flattened to approximately 1V on 2H by systematic drilling and blasting. The product was a mixture of sound rock and firm rock that was wasted, since the need for these materials had been met. (Contractor was permitted to process some for use as road plating material.) The result was a "stair-step" foundation on the east and west slopes. The bottom of the channel was excavated from elevation 330.0 to 315.0 approximately. The hanging wall of the fault was further excavated to elevation 309.0 after grouting had been completed. The fault proper was taken down to elevation 305.0 for a width of approximately 10 feet and this "trench" was then backfilled with concrete and the entire portion of the diversion channel underlying the impervious zone of the embankment received shotcrete (refer to Appendices D thru F).

7.1.2-EE Drilling and Grouting: Following cleanup, inspection, and approval, the area was released to a comprehensive drilling and grouting program. The curtain grouting consisted of a single line on the slopes and two lines across the bottom of the channel. Grouting was performed to a depth of 125 feet below the surface in 3 zones using stage-grouting. Significant surface leaks were noted in the area west of the fault in highly weathered rock, even though 3 to 5-foot nipples were used. Consolidation grouting was performed in the area downstream of centerline and west of the fault (Plate 109). Grouting was taken to a point below the fault, using stop-grouting. Two consolidation grout lines were added on the west side and one contract line deleted on the east side. Full details of consolidation grouting are covered later in this section.

7.1.3-EE Filters: Plans called for the downstream side of the cutoff trench in the bottom of the channel to receive filters and shotcrete. During the inspection on 5 May 1982, it was recommended that this configuration of filters be re-evaluated and consideration be given to shotcreting the downstream wall. SASEN-GS later directed that the downstream wall from elevation 310.0 to 330.0, approximately, be shotcreted and the filters begin at the 330.0 level.

7.1.4-EE Consolidation Grouting: The hanging wall of the Diversion Channel Fault downstream of the main grout curtain was consolidated by stop grouting. Plate 109 shows the grout hole layout profile illustrating the geometrical relationship between the consolidated zone and athe fault plane.

Stop grouting (as opposed to stage grouting) was the method chosen for this work as it was believed to be more effective for consolidation. Table 9 lists the grout takes and provides a statistical analysis of the grouting operation. Exploratory borings taken after grouting indicated that the grouting program was effective and that the desired end produce was obtained.

7.2-EE Soft Pockets Near Diversion Channel: Around the centerline of the embankment between Stations 12+00 and 13+00 were several pockets of weathered rock. This rippable material was similar to, but stronger than that encountered in the bathtub area. It differed from the bathtub in that

TABLE 9

CONSOLIDATION GROUTING

HOLE NO.		DEPTH(feet)	TOTAL TAKE(cu. ft.)	TAKE/LF(cu. ft.)	REMARKS
U-1		25.0	0.0	0.0	
U-2		25.0	0.0	0.0	
U-3		25.0	0.09	0.004	
V-1		25.0	4.18	0.167	
V-2		25.0	0.0	0.0	
V-3		25.0	0.0	0.0	
W-1		45.0	6.30	0.140	Surface Leak
W-2		45.0	3.54	0.079	Comm w/V1, W3
W-3		45.0	0.36	0.008	
X-1	1/2	40.0	17.84	0.446	
	1/1	65.0	5.12	0.205	Surface Leak
X-2	1/2	30.0	2.71	0.090	
	1/1	65.0	0.28	0.008	
X-3	1/2	30.0	0.14	0.005	Surface Leak
	1/1	65.0	3.42	0.098	
Y-1	1/2	45.0	0.36	0.008	Surface Leak
	1/1	85.0	0.72	0.018	
Y-2	1/2	45.0	7.36	0.164	Surface Leak
	1/1	85.0	0.72	0.009	
Y-3	1/2	45.0	0.09	0.002	
	1/1	85.0	0.00	0.0	
Z-1	1/3	40.0	19.33	0.483	Surface Leak
	1/2	65.0	5.0	0.200	
	1/1	125.0	2.67	0.045	
Z-2	1/3	40.0	0.27	0.007	
	1/2	100.0	0.09	0.002	
	1/1	125.0	0.0	0.0	
Z-3	1/3	40.0	0.18	0.005	
	1/2	100.0	0.0	0.0	
	1/1	125.0	0.0	0.0	

TABLE 9 (CONTINUED)

CONSOLIDATION GROUTING

HOLE NO.	DEPTH(feet)	TOTAL TAKE(cu. ft.)	TAKE/LF(cu. ft.)	REMARKS
ZZ-1 1/3	60.0	0.45	0.008	
1/2	90.0	0.0	0.0	
1/1	125.0	2.45	0.070	
ZZ-2 1/1	125.0	5.18	0.041	Comm W/ZZ-3
ZZ-3 1/1	125.0	0.45	0.004	Surface Leak
V-1.5	25.0	0.09	0.004	
W-2.5	20.0	0.0	0.0	
	45.0	0.0	0.0	
X-1.5	40.0	0.18	0.005	
X-2.5 1/2	40.0	0.0	0.0	
1/1	65.0	0.18	0.007	
Y-1.5 1/3	50.0	0.18	0.004	
1/2	80.0	0.0	0.0	
1/1	105.0	0.18	0.007	
Z-1.5 1/3	40.0	0.18	0.005	
1/2	80.0	0.09	0.002	
1/1	105.0	0.09	0.004	

TOTAL GROUT HOLES DRILLED: 1,915 LF

TOTAL GROUT TAKE: 90.11 cu. ft.

MEAN GROUT TAKE: 0.047 (cu. ft./LF)

STANDARD DEVIATION: 0.103

it was not saturated with groundwater at the time it was grouted. Although core holes were drilled in and near these somewhat jointed areas, the excessive weathering could not be related to an unusual geologic feature. Three extra lines of 20-foot deep holes were grouted upstream from Stations 11+80 to 13+00 to assure seepage cutoff in this rock.

7.3-EE "Bathtub": An area of severe weathering of the foundation rock was encountered in the Georgia west section from approximately Station 5+50 to Station 6+50 (see Plate 73). This area is the convergence point of three dikes resulting in weathering far greater than the surrounding area. Station 5+50 was the end point of the transition trench slope; therefore areas upstation were specified to be taken to ripper refusal. Since ripper refusal was being approached in surrounding areas, the contractor was directed to continue in this area, although the "design grade" had been passed. Since there were rock knobs above "grade" being encountered routinely, the possibility of not being able to obtain rippable rock in this section was not determined until the excavation was near elevation 395, about 10 feet below "grade". ("Grade" is taken as that elevation shown on Plate EE-25 of the Contract Plans, an anticipated elevation of ripper refusal.) At that time, an inspection was made of the area by field, district, and diversion personnel. On 16 May 1980, following cleanup of the area, and obtaining several exploratory borings, the decision was made to stop excavation and begin treatment. Since the area was seeping a considerable amount of water, some method of controlling the water was necessary during fill placement. A system of pumps and a concrete cap was agreed upon and the area released to drilling and grouting. A triple grout line was directed to be used and the grout curtain was shifted upstream to coincide with one of the dikes. The shift and triple line ended near Station 7+00. Additionally, a single line to a depth of 25 feet was installed along the upstream edge of the "bathtub" to help control seepage (ineffective). Drilling and grouting were completed in this area in late November 1980.

Details for the special foundation treatment for the "Bathtub" area were contained in Modification H (P017). In June 1983, the contractor began "mucking out" the accumulation of water and mud from the area. Since this was the lowest point in the section and located at the toe of the Georgia transition slope, a considerable amount of material had settled here. The tedious work was slowed by rain and additional wash-in of material. The base rock was very weathered and deteriorated quickly. Final preparation was performed only hours prior to concrete placement. Treatment of the area was as follows:

a. Final Cleanup and Wash-down: This resulted in an overall average foundation elevation of 385.0 feet.

b. Seepage Control: A trench was dug along the upstream side and a 36-inch corrugated metal pipe installed vertically at each end (No. 1 and No. 2). Numerous 1-inch holes were burned in the pipe to permit water flow into the sump. A similar sump (No. 3) and trench were located on the downstream side of the area over a flowing seep. The trenches were then filled with 3-inch concrete aggregate. An additional sump (No. 4) was installed in a similar manner upstream of the "bathtub", not connected to the other sumps, needed for other seepage control.

c. Concrete Pad: The rock drain, above, was then covered with plastic and the area covered with concrete (the trenches did not connect) with a thickness of approximately 3 feet. A total of 114 cubic yards of concrete was used. The top of the concrete pad was not level, but had an average elevation of about 389.0.

d. Sand Drain: Water flowing out of the upstream slope remained to be controlled. After the concrete had cured, the plastic was removed from the top of the concrete aggregate and approximately 1 foot of aggregate removed. This was replaced with 3 to 6 inches of coarse filter. This was followed with filter sand. The sand was saturated and compacted beneath the top of the pad. A 12-inch wide sand drain was placed adjacent to the upstream slope to intercept seepage and carry it to the sumps. Impervious fill was placed on the concrete pad. The sand drain extended to elevation 395.0.

e. Fill Placement: Following the treatment, fill placement continued until drainage out of the area was obtained, approximately 2 days work in August and November 1981. Work in the area then ceased until June 1982. Pumping of the sumps continued during this time since the fill level did not exceed the piezometric head.

f. Capping: During fill placement in the summer of 1982, the piezometric level in the sumps was monitored by cutting the pumps off and allowing water to rise in the sumps. When the fill reached elevation 415.0, it was determined that the sumps were no longer needed and could be sealed. Grout pipes had been installed in the trenches for grouting purposes. A 1:1 grout mix was pumped into the grout pipes (gravity, no added pressure) until grout showed in the sumps. This was allowed to take its initial set, then the sumps were filled with concrete (total 32 cubic yards).

g. Summary Data:

SUMP	BOTTOM OF TRENCH	TOP OF CONCRETE	TOP OF SAND DRAIN	TOP OF FINAL RISER	BAGS OF CEMENT GROUT
1	383.3	388.6	395.0	417.5	57
2	383.0	388.2	395.0	418.0	82
3	384.2	390.2	N/A	418.1	102
4	390.5	N/A	395.0	418.1	114

7.4-EE Relief Joints: There were several clean, unfilled relief joints on the Georgia abutment under both the concrete dam and the embankment. These sheet joints separated otherwise sound foundation rock into several layers like an onion. When the Georgia east portion of the Georgia embankment was excavated, the contractor's bulldozer ripped along these sheet joints for some distance. When it was noted that two of the joints had been chased to some depth, it was decided to end ripping and excavation at a cross-dike near the 22+00 station of the dam axis. At this point, there was a 10-12-foot stepup on a steep (70-80 degree) slope. After the area was grouted, an unformed fillet was placed to about a foot above the upper joint, furnishing a low angle face against which fill could be easily

compacted. Grout communication was noted in the first zone between holes 200 feet apart below this face, probably indicating another relief joint at a shallow depth.

7.5-EE <u>Formed Concrete Fillet - South Carolina</u>: Approximately 7,500 cubic yards of rock excavation was required in South Carolina to remove a rock knob in the cutoff trench. The rock on the upstream wall was characteristically blocky, jointed, loose, and had numerous clay seams. The blasting left a very rough blocky, upstream slope having large, high overhangs and open seams throughout the blast zone. Due to the geology of the area and the presence of clay seams, it was felt that the jointed nature of the rock was not caused by blasting, although some of the seams were probably opened further. Treatment considerations included additional excavation in the slope, pulling loose blocks out of the slope, and placing a formed fillet to cover the area. Since any excavation attempt may have accentuated the problem, the decision was made to use a formed concrete fillet, approximately 15 feet high. The fillet was placed in two lifts, with dowels, and had a total of 244 cubic yards of concrete. This was accomplished under Modification P012 (L).

7.0-PH SPECIAL OR UNUSUAL CONDITIONS DURING CONSTRUCTION - POWERHOUSE

The foundation of the Russell powerhouse was found to be rather straight forward from a geologic point of view and no unusual nor unexpected conditions were encountered during construction.

8.0-CD FOUNDATION ANCHORS AND ROCK BOLTS - CONCRETE DAM

Rock anchors were installed on both the excavation and concrete dam contracts. The anchors on the excavation contract were for prevention of slides as described above. Foundation anchors and wall anchors were installed on the concrete dam contract.

The rock anchors installed under the excavation contract were Dywidag threadbars of 1-inch diameter and 60,000 psi yield strength. They were installed in 1-5/8-inch diameter holes with two 12-inch long Celtite fast set, two-component, polyester resin cartridges in the bottom of the hole. After the resin was allowed to set, the anchors were tensioned with a torque wrench to 2/3 of the bar yield strength (21,500 lf.). Tension was locked into the bolt by a 1-7/16-inch hexagonal nut against a 6" x 6" steel plate 3/8 inch thick. Corrosion protection for the stressing length was not provided.

Foundation anchors were installed in only one of the thirty-two monoliths of the concrete dam. Block 26, unlike the others, was divided into two parts by a construction joint. The downstream part of the block formed the flip bucket for the easternmost tainter gate. Since the flow of water over the flip bucket would impart an overturning moment to this monolith, it was considered necessary to anchor the concrete firmly to the foundation rock. This was done by grouting number 11 reinforcing steel bars into 10-foot deep holes in the downstream area of Block 26. The projecting ends of the bars were deformed into a J-shape prior to installation and the concrete was placed around them after the grout cured. The broom grout mix that was utilized to treat the foundation prior to concrete placement on rock was used to cement the bars into the holes.

The rest of the anchors installed on the concrete dam contract were used to secure the tailrace and erection bay hang-on walls. Two types of anchor were used. Twenty-seven one-inch diameter billet steel bars were used as untensioned dowels in the lower part of the tailrace wall. These dowels were installed 3 feet into rock using Celtite fast set resin cartridges prior to placement of the concrete wall. Hooked attachements project into the wall to about 6" from the outer surface of the wall.

On the upper part of the tailrace wall and on the erection bay wall, post-tensioned anchors were used. There were 145 of these in the erection bay and 117 in the tailrace wall. The tailrace wall anchors were originally designed to be installed before placement of the wall, but proved to be difficult to install in the very irregular rock face. The contractor proposed, and the Corps accepted, a no-cost change to modify the contract to allow forming of blockouts in the wall and installation of the anchors in these blockouts after the wall was in place. Thus, the same installation procedure was used on both walls.

The anchors were 1-1/4-inch Dywidag threadbars of 187,500 lb. ultimate strength. Lengths varying from 24 feet (20' in rock) to 35 feet (31' in rock) were installed in holes of 2-inch diameter. A combination of fast and slow set Celtite cartridges was inserted into the hole to give full encapsulation with a 12-foot (minimum) anchor zone. After insertion of the thread-bar and setting of the anchor zone resin, the bar was stressed to

150,000 lbs. and locked off at 112,000 lbs. by a hexagonal nut against a 9' x 9' x 1-3/4 anchor plate. When all the anchors had been installed in a wall, the contractor flame-cut the anchor 3"-6" inside the blockout and filled the blockouts with concrete. Flame-cutting was allowed only after the contractor proved that he could cut the threadbars with a cutting torch with less residual heat than that involved in sawing them with an abrasive disc. Plate 128 contains installation details of rock anchors used in the tailrace and erection bay hang-on walls.

8.0-PH FOUNDATION ANCHORS AND ROCK BOLTS - POWERHOUSE

8.1-PH General: A relatively large number of rock support elements were employed in the powerhouse structure so that structural stability criteria were met. Both deformed bar rock bolts and multi-strand, high-capacity rock anchors were utilized in meeting these criteria.

8.2-PH Deformed Bar Rock Bolts: Deformed bar rock bolts (also referred to as "thread bar rock bolts") constitute, by far, the majority of the rock support elements installed in the powerhouse. These bolts are 1 1/4 inches in diameter and are various lengths, grade 150, and utilized a polyester resin grout. Each rock bolt assembly consists of the bolt, a steel bearing plate, a hexagonal nut, a sufficient number of fast set resin cartridges for the anchor zone, and sufficient number of slow set resin cartridges to fill the remainder of the drill hole. A typical installation sequence is given below:

1. Drill and clean hole (percussion drilling only was allowed).

2. Insert the required number of fast set and slow set polyester resin grout cartridges.

3. Insert the bolt into the hole, using the air-track drill to apply both rotation and downward pressure.

4. Once the bolt is inserted to the required depth, continue rotation until the anchor zone grout sets up.

5. Remove the drill, install the hydraulic stressing jack and stress the anchor with the required amount of force.

6. Tighten the hexagonal lock-off nut and remove the stressing jack.

Plates 127 and 128 show the locations of the deformed bar rock bolts and Plate 129 shows typical details.

8.2.1-PH Draft Tube Slab and Service Bay Slab Rock Bolts: Plate 127 shows the locations for these bolts. These areas utilized 35-foot long bolts except in selected areas in the vicinity of the unwatering drainage trench where longer bolts were required so that the 20-foot anchor zone in rock criteria would be met.

8.2.2-PH Design Requirement for Anchoring the Draft Tube and Service Bay Slabs: Each generator bay monolith was designed as a "stand alone"

gravity structure with the dead load of each monolith being of such magnitude that all anticipated hydrostatic uplift pressures would be compensated for and hence, no external anchorage would be required. However, the draft tube and service bay slabs are not structurally integrated with the monoliths and do not transmit foundation loads, contribute weight, nor resist uplift forces. Therefore, a system of foundation drains (described in Section 10.0-PH and shown on Plate 126 and rock bolts (Plate 127) was used to overcome hydrostatic uplift forces within these areas. The following design assumptions were made in the layout of the rock bolts for these areas:

a. Foundation drains were assumed to be 50% effective.

b. The maximum expectable tailwater elevation was assumed to be Clarks Hill standard project flood (elev. 342.0).

8.2.3-PH Erection Bay Rock Bolts: Deformed bar rock bolts were used in both the erection bay slab (vertical, 35-foot lengths) and the erection bay substructure wall (horizontal, variable lengths) to compensate for hydrostatic pressures acting against these structural elements. Plates 127 and 128 depict the spatial arrangement of bolts in these portions of the structure. The erection bay substructure wall and tailrace hang-on wall was built under the concrete dam contract by the concrete dam contractor (Dravo-Groves). Initial plans for the substructure wall and tailrace hang-on wall called for the installation of rock bolts directly against the rock face. The contractor submitted a proposal to install the bolts through the concrete wall; this proposal was evaluated and accepted by the Corps and was incorporated into Modification FF. Plate 128 depicts the locations and details of these bolts.

8.2.4-PH Spillway Training Wall: Rock bolting and anchoring for the spillway training wall is covered in detail in Section 4.0-PH.

8.3-PH Multi-Strand, High-Capacity Rock Anchors: Rock anchors are used in the powerhouse in Units 5 through 8 and in the spillway training wall. Thirty strand anchors are located in the units while twenty-three strand anchors are to be found in the spillway training wall. Each strand is 1/2" in diameter and is composed of 7 wires conforming to ASTM A416 for prestressing steel. Each strand possesses a minimum ultimate strength of 41.3 kips. Installation procedures for both types of anchors are similar with only the ultimate and lock-off applied forces differing. Plate 129 depicts typical as-installed details while Plate 127 depicts the locations.

8.3.1-PH Installation Procedure: The following is the installation sequence for all of the stranded rock anchors employed in the powerhouse:

1. Drill holes with down-hole hammer.

2. Pressure test holes, grout, and re-drill if necessary.

3. Insert anchor and perform first-stage grouting.

4. Test grout cylinders for strength (> 3000 psi).

5. Stress anchors to 80% of ultimate strength hold for 10 minutes and lock-off at 70% of ultimate.

6. After 7-day waiting period, perform lift-off test. If lift-off test reveals no relaxation, got to step 7. If lift-off indicates that relaxation has taken place, go back to step 5.

7. Perform second-stage grouting.

8.3.2-PH - Grout Materials: The use of a high-early strength, non-shrink, Portland cement-based grout was specified for the stranded rock anchors. The contract specified a site-mixed grout using a number of admixtures, but the contractor proposed the use of a pre-mixed, commerically available grout. After evaluation of the manufacturer's literature and limited testing at both the Project Office and the SAD Lab, approval was given to the contractor to use the commercially available grout. This approval was contingent upon satisfactory performance and at no additional cost to the Government. During the installation of the anchors using this grout, no problems were encountered. It is felt that performance probably exceeded what would have been attained had the original grout mix been used.

8.3.3-PH Engineering Considerations for Use of Stranded Rock Anchors in Units 5 Through 8: As stated previously each powerhouse monolith was designed as a stand-alone gravity structure as the total dead load of each unit was sufficient to prevent hydrostatic uplift. However, since Units 5-8 will consist only of the skeletonized structure for a number of years, it was imperative that the reduced dead load of these units be compensated for so as to prevent uplift. A number of different schemes were analyzed for sufficiency, constructability, maintainability, and cost effectiveness. On all counts, the use of rock anchors proved to be the most viable alternative. Although the time span in which the anchors are required to function is relatively short (less than 10 years), they are a permanent part of the structure and should function properly for the entire life of the project.

8.3.4-PH Stranded Rock Anchors in the Spillway Training Wall: Rock anchorage requirements and considerations for the spillway training wall are covered in detail in Section 4.0-PH.

9.0-CD CHARACTER OF FOUNDATION - CONCRETE DAM

9.1-CD Foundation Surface: After foundation preparation, most of the Richard B. Russell foundation surface was very irregular. Exceptions to this were the parts of Monoliths 2 and 3 which were excavated with a backhoe along a low angle joint, and Monolith 26, excavated along a clay seam. These areas were fairly smooth after drummy plates and slabs of rock had been removed.

The extensive jointing, faulting, and shear zones in the dam area, along with the multiple dikes and dike swarms gave the rock a tendency to break in an irregular, blocky manner. This was intensified by the release of high velocity gasses from blasting agents and explosives used in modern construction practice. The combination of hard, strong rock with pre-existing, weaker planes made it difficult to achieve regular excavation surfaces, even when presplitting techniques were used.

Design slopes flatter than 1V on 1H usually broke below grade in a blocky or stairstepped manner. This was probably due to the presence of incipient near-horizontal planes of breakage in the foundation rock. These planes may be created by stress relief or tectonic forces, but they are present in most rock masses in the area, including the Elberton Granite. When explosives were detonated, these planes furnished an easy exit for expanding gasses. Holes downslope propagated breaks well into and through the plane of the design slope. When final rock removal had been accomplished to remove rock which had been loosened, the effect was even more pronounced. Areas where the stairstep effect was most apparent were the 1V on 3.5H slope downstream of Monoliths 8 through 16, the 1V on 3.18H slope downstream of Monoliths 6 and 7, and the sump in Monolith 9 (excavated by Dravo-Groves).

Where the design surface was horizontal, the final surface was also uneven. Preferential breakage along the natural discontinuities described above resulted in a foundation with many abrupt changes in slope. The felsic rocks were more resistant to breakage than were the mafic dikes. Percussion tools such as hoerams and pavement breakers were used as little as possible in the brittle metadiabase. Another factor which caused an irregular surface was overdepth blast holes. Although the contractor maintained good drilling control on most blasts to grade, some holes were drilled below specified tolerances. During loading, holes determined to be overdepth were bottom-stemmed back to grade and loaded as planned. However, a few of these holes were not identified. Shooting an overdepth hole resulted in a depression, usually conical, in the foundation.

Foundation surfaces 1V on 1H or steeper were commonly more irregular than what is usual for presplit slopes. The rock in these faces broke along joints at varying attitudes. The presplit face between Monoliths 7 and 8 was the only one between monoliths which was excavated close to design. The faces upstream of Monoliths 8 through 13 also broke out well. Some of the smaller vertical faces between monoliths broke irregularly along their entire length. It was necessary to make "plug pours" in several of the monoliths on the east abutment to replace rock broken off vertical faces between monoliths.

9.2-CD Condition of Foundation Rock: The Richard B. Russell concrete dam was founded entirely on rock. With the exception of a few shear zones which were hydrothermally altered or filled with softer minerals, the rock was sound. The engineering characteristics were within those reported in the Design Memorandum 8 - Geology and summarized in Section 3.5 of this report.

9.3-CD Water: The measures described in Section 5.0-CD of this report were adequate to allow excavation of the dam and powerhouse in the dry. Some water entered the excavation from upstream of the intake and spillway blocks, but the quantity of flow was too low to cause problems. During concrete placement, water problems were encountered in the intake and spillway sections west of Block 24. Blocks 1-7 and 24-32 were founded at higher elevations. Seepage and natural runoff were intercepted and ponded by a low dike/road 20 to 30 feet upstream of Blocks 10-23. The water level here was probably at or above 300 MSL.

In Blocks 8-23, where seepage along joints or seams was very slight, concrete was merely placed and vibrated well against the seam. In these cases, the weight and strength of the concrete was judged to be sufficient to stop these minor leaks without special treatment. Blocks in which no special treatment was needed were Block 8 (where wet weather seeps were present along the 7/8 face), Block 10 (a few seeps were present on the upstream face), Block 11, Block 17, Block 20, and Block 21.

Where seepage was present along the upstream face at fairly low pressures, visqueen sheeting was used to separate the water seeps from the concrete. This was done by placing the visqueen on the face immediately before the last buckets of concrete were placed in the block. (Placement proceeded from downstream to upstream). Then the last concrete was placed against the visqueen, causing the water to rise at the upstream face. This was done in Blocks 9, 14, and 18.

Upstream of Blocks 13, 14, and 15, a small sump was created by a backhoe to collect water. The sump was pumped nearly dry during placement. This eliminated the necessity for special treatment in Blocks 13-15.

Where moderate flows were present on the upstream face, french drains were made by pinning 6" wide steel strips vertically to the foundation approximately 1 foot from the face. These were filled with uniformly graded 3" aggregate and a visqueen sheet was placed over them. Pipes, or 55-gallon drums, were used to remove or pump water from these drains. At least 1 supply and 1 return pipe were always installed.

After one or more lifts of concrete were in place and cured, the drains were pressure grouted with 1:1 grout until grout was seen in the return lines. Blocks 12 and 22 were treated this way until grout was seen in the return lines. Blocks 19 and 16 had spot flows. Block 16 was the only monolith with a leak in the horizontal foundation. This leak was west and downstream of the inverted plumb bob, below the present inspection gallery. These spot flow areas were surrounded or filled with clean 3" gravel with 55-gallon drums, used to blockout a space for grout pipes. They were treated as the french drain above, but after being confined by one or more lifts, they were first grouted, then concreted up with a 3/4" mix.

Locations of Treatment Described:

Block 12 - Against the eastern 2/3 of the upstream face, relief pipe at the corner near the 12/13 joint.

Block 16 - Immediately below the inspection gallery as the gallery turns a corner to enter Block 17, near the inverted plumb bob.

Block 19 - Against the upstream face about 10 feet from the 18/19 joint.

Block 22 - Against the upstream face in the western corner.

9.4-CD Special or Unusual Conditions: Foundation conditions which required special consideration and/or treatment in the construction of the Russell Dam include seams, unexpected areas of deteriorated rock, and water problems.

The clean joints in the Georgia abutment have been mentioned in the Foundation Exploration and Geology sections of this report. They were discovered by examining 4-inch core taken under the excavation contract. The joints were low angle, irregular, and leached. The air-trac exploration described in the Exploration Section was initiated to determine the extent and attitude of these joints. It was discovered that there were (probably) three seams, surfacing in Monoliths 5 or 6, Monoliths 2 and 3, and outside the dam under the Georgia embankment, and exiting the stepup face between Blocks 7 and 8. The most probably dip on these subparallel joints was toward the river and slightly upstream. Because they were clean, irregular (with good rock to rock contact), and dipped upstream, it was decided that these joints posed no problem to the integrity of the dam, except that if ungrouted, they would allow water to seep through the foundation. They were penetrated by curtain grout holes, and based on communication in the first zone, were grouted up. No further treatment was considered necessary.

An unexpected area of deteriorated rock was discovered during excavation of Monoliths 2 and 3. This rock, located between one of the joints mentioned above and a shear zone, showed evidence of hydrothermal alteration and severe weathering. Possibly, the joint allowed easy penetration by hydrothermal solutions from the (also hydrothermally altered) shear zone which caused decomposition of the rock and allowed (later) rapid weathering. The rock below the joint was hard, fresh, and only slightly weathered, so treatment beyond removal of the decomposed material and replacement with concrete was considered unnecessary.

A (probable) relief joint intersecting the upstream face of Blocks 13, 14, and 15 allowed ponded water upstream to infiltrate into the excavation, depositing rusty-appearing iron salts on the foundation and causing minor water problems during construction of these blocks. The water was handled during placement as described in Section 10.0-CD of this report. The iron salts were cleaned from the foundation by sand-blasting and/or wate-- blasting at maximum force with great difficulty.

Another area of deteriorated rock which had to be removed and replaced was in the upstream part of Monolith 16 near the inverted plumb bob located in that block. The hydrothermally altered and weathered rock was above a

low angle shear zone. The area was excavated to sound rock. The geology and topography of that zone can be seen on the maps of the Block 16 area.

A foundation condition which caused considerable construction change was the presence of seams filled with shattered and decomposed rock in the spillway area. These seams were noted while logging core drilled on the excavation contract. Excavation grades were changed to remove rock above some of these seams, but some were left after the excavation contract was completed. Except where they occurred under trestle footers, the rock above them was removed under the concrete dam contract as part of the rough cleanup phase of foundation preparation. Where trestle footers were underlain by seams, the seam and the rock above it were removed to within a few feet of the footer. A small amount of rock was left to avoid undermining the footer. In Block 17, a 20' x 10' slab of rock was left under the northeast trestle footer. In Block 19, rock was removed to within 1 to 5 feet of the edge of the northeast footer, then the footer was surrounded by a concrete collar to protect it until the first lift of concrete could be placed.

During foundation rock removal in Monolith 22, a seam filled with mud and rock fragments was discovered. This seam was chased into Monolith 23, where it was tighter and contained no mud. Previous borings indicated that the seam stopped at a dike which crossed the monolith diagonally a short distance from the existing face. It was decided to discontinue rock removal at the point where there was no mud and there were fewer rock fragments, and treat the seam by grouting. This treatment is described in Section 10.1-CD.

Clay filled breaks in foundation rock also necessitated changes in construction. These "clay seams" were joints, relief joints, and irregular breaks at all orientations which were filled with a plastic, red clay. Analysis of the clay showed that it was primarily kaolinite, probably a product of chemical weathering of feldspars. It had probably been transported to the "seams" by the movement of groundwater, and then had settled out of suspension in all available open spaces. The "clay seams" were present in both abutments, but were a problem only in the South Carolina abutment where foundation grade had been set higher in the zone of weathering. In this abutment, the extensive, near-horizontal relief joints were the major problem. Grades were lowered to eliminate the seams and rock was removed under modification to found below them. Blocks 28, 27, and 26 were those overexcavated. Additional changes which were made to adapt the structure included additional grout holes, a change in the geometry of the inspection gallery, additional instrumentation, and a more extensive system of waterstops on the monolith construction joint between Blocks 28 and 29.

A final problem which caused minor difficulty was the presence of near vertical round-shouldered rifts filled with cobbles, sand, and gravel in the riverbed. These rifts were probably joints which had been widened by the action of the river's current and bedload on the rock adjacent to the existing open joints. The loose material in the rifts tended to fill in blast holes as they were drilled, causing the excavation contractor's blast holes to collapse as they were drilled.

Prior to any concrete placement for dam construction, a written record of final foundation approval was made after inspection of the foundation in

the field. Placement cards were kept for each monolith; however, these were not available for inclusion in this report.

9.0-EE CHARACTER OF THE FOUNDATION - EMBANKMENTS

9.1-EE Foundation Surface: The foundation surfaces upon which fill was placed can be categorized as one of the following: (1) in-situ residual soil in the transitions; (2) shotcrete or dental concrete treated firm rock and fractured sound rock; or, (3) unfractured sound rock. For details on the treatment of the various areas see Section 10.0-EE of this report.

9.2-EE Engineering Characteristics of Foundation Materials: Section 3.5 of this report summarizes the engineering characteristics of the foundation soil and rock for the embankments. Design Memorandum 8 - Geology and Design Memorandum 17 - Earth Embankments contain detailed studies and test results for the foundation and embankment materials for this project.

9.3-EE Groundwater Conditions Affecting Construction: Groundwater conditions affecting construction of the embankments were only of local extent, and with the exception of the "Bathtub", presented no major problems. Section 7.0-EE contains details on the "Bathtub" area and Section 5.0-EE contains details on dewatering in general. Also, refer to the Embankment Performance and Criteria Report for a more exhaustive treatment of dewatering of the foundation.

9.0-PH CHARACTER OF FOUNDATION - POWERHOUSE

9.1-PH Foundation Surface: The surface of the powerhouse foundation is very irregular due to the intense jointing of the rock. Joints within the foundation rock assume two major geometric styles which are:

a. High-Angle, nearly vertical joints with two predominant strike directions. These joints have spacing that ranges from about 2 inches up to about 4 feet with an average spacing of about 1 foot.

b. Subhorizontal Relief or Exfoliation Joints. These joints are subparallel to the topographic surface and were probably caused by removal of the overlying confining material by erosion and/or excavation. The spacing of these joints, many of which were only incipient until subjected to excavation equipment, ranges from about 2 inches to about 4 feet with an average spacing of about 1-1/2 feet.

As may be interpreted from the preceding paragraph, the powerhouse foundation consists of a "stair step" arrangement of joint blocks in those areas where the foundation contains a slope, and is more or less flat lying in the remainder. The "stair step" arrangement is particularly noticeable in the draft tube area of the powerhouse where the foundation slopes upstream and rather than being a detrimental characteristic, is in fact positive in that it increases the sliding resistance of the structure.

9.2-PH Engineering Characteristics: The powerhouse is founded on fresh unweathered rock. The rock is a hard, dense metamorphic rock whose strength and other engineering characteristics are more than adequate for the

structure being built upon it. For a more complete description of testing performed, refer to Section 3.5 and Table 3, page 28.

9.2.1-PH Additional Physical Tests: In addition to the rock tests mentioned in Section 3.5, tests for suitability of the rocks from the damsite for concrete aggregate were also performed. Although performed for another purpose, an analysis of the test results leads one to conclude that no adverse chemical reactions between the rock foundation and the concrete of the structure should take place. Further discussion of these tests are presented in detail in Design Memorandum 12 - Construction Materials.

9.3-PH Groundwater: The occurrence of groundwater at a given site is a function of many interrelated factors, but perhaps the most important parameter is the permeability of the geologic formations present at the site. The Russell powerhouse is founded entirely in crystalline rocks whose primary permeability is very low and whose secondary permeability is also low due to the fact that most of the discontinuities occurring in the rocks are either tight, healed, filled with impermeable clays, or are discontinuous and not interconnected. Therefore, groundwater flows did not cause any great problems during construction of the powerhouse. Those groundwater inflows which were present were relatively small, localized affairs, which were controlled by french drains, described in Section 10.0-PH of this report. Foundation relief wells and weep holes are also described in Section 10.0-PH.

9.4-PH Special or Unusual Conditions: The foundation of the Richard B. Russell powerhouse was found to be rather straight forward from a geologic point of view and no unusual nor unexpected conditions were encountered during construction.

10.0-CD FOUNDATION TREATMENT - CONCRETE DAM

10.1-CD Grouting Prior to Concrete Placement: A 1" to 4" thick seam filled with mud, decomposed rock, and rock fragments was exposed in Block 22 and during rough cleanup in that block was chased into Block 23. The intended method of treatment was to remove the seam and rock above it. However, as this excavation proceeded deeper into Monolith 23, the seam proved to be tighter. Fresh rock chips, such as are produced by blasting, were the only filling. Boring logs indicated that this seam probably did not extend past a metadiabase dike which crossed the block diagonally about 25 feet from the existing face. Due to the change in thickness and character of the seam, it was decided to leave the 4 feet of sound rock above the seam in the foundation and grout the remaining open portion.

Modification P023 was issued to provide for this operation. The contractor was directed to:

1. Drill fifteen 3" diameter holes through seams.

2. Wash holes and intersected seam until wash water return is clean.

3. Grout 1-1/2" diameter pipes into the holes.

4. Construct french drains at the excavation face. Install 2-1/2" diameter return pipes.

5. Fabricate and install "Box Type" uplift cell in lieu of cell #24.

6. Place plug pour over box cell and french drain.

7. Clean surface of affected area to allow observation of joints and fissures.

8. Using a 1:1 mix, begin grouting pipes as directed. If grout return from any hole is noted, cap off that hole. When maximum pressure of 15 psi is reached and held, cap off that hole and connect to next hole as directed. Continue until return or refusal is obtained in all holes and 2-1/2" return pipes.

9. Four (4) hours after grouting, wash out uplift cell holes to full depth.

10.2-CD Curtain Grouting: The concrete dam grout curtain is a single line, three zone curtain with holes spaced 5 feet apart. A few quaternary holes were drilled 2.5 feet away from existing holes where communication or high takes warranted. These holes were drilled through 3" sleeves (positioned prior to concrete placement) angled 15 degrees from vertical in an upstream direction perpendicular to the dam axis. The uniformity of the curtain as described was interrupted in three places. At both ends of the dam, grout holes were fanned out to provide for overlap with the curtain grout holes under the earth embankments. These holes were drilled at variable angles in a vertical plane along the dam axis. Spacing on these holes was two to four feet. They are depicted on Plates 51 and 52. The grout curtain in Monolith 28 was also irregular. This monolith was lowered

drastically during excavation, while Monolith 29 was left at original design grade. The grout curtain in this area would have had a large "window" where the collar elevation of the grout holes stepped up over 40 feet if the original design were followed. The inspection gallery was redesigned to include a "T" shaped appendix, with the crosspiece of the T running parallel to the 28-29 stepup face. Horizontal, 20 degree, and 45 degree holes were used to overlap the curtain holes originating in Block 29. Three holes were added on the stair from the original inspection gallery to Monolith 29. This is depicted on Plate 52.

Construction of the curtain was done from a working (inspection) gallery approximately 5' above the design rock foundation. The sequence of work was to drill and grout holes on 20-foot centers in the first zone (40' deep), then to split with secondary holes 10' away in the same zone, then to split with tertiaries 5 feet away. The second zone (80' deep) was completed in a similar manner with primary and secondary holes, and the third zone (120' deep) was completed with primary holes only. These holes were further split on either side when the hole communicated with others over a considerable distance or where the hole took ten bags or more of grout in that zone.

Grout holes were drilled with air-driven CP-65 drills using diamond-set coring bits, 1.5 inches in diameter, with water as the drilling medium. The metadacite and quartz porphyry were very hard, drilling more slowly than western (U.S.) quartz monzonites and granites according to drillers who had worked in these rocks. Rates for the first 5 months of grout hole drilling averaged 12 feet of redrill (through the cement-bentonite mix described below) and 36.2 feet of rock in an 8-hour drill shift. The original face-set bits used often lasted only 4-10 feet, so the contractor went to an impregnated coring bit to prolong bit life. This did not, however, improve drilling rates much.

After drilling, the holes were washed and pressure tested. When a hole had been drilled to full depth it was washed for a short time through the drill string to get the cuttings out of the hole. Following this, the holes were pressure-tested with water for 5 minutes, using the maximum allowable pressure. If the hole communicated with another, it was washed until the exhaust water was clear.

When holes within a specific reach of curtain had been drilled, washed, and pressure tested, grouting operations began. Bentonite and water were mixed in a paddle-type mixer located outside the dam, then routed through a Galigher centrifugal pump to thoroughly mix and hydrate the bentonite. Cement was then added to the water-bentonite mix in the same mixer. From the mixer, the grout was dumped into a large (30+ cu. yd.) sump. A constantly-circulating grout line (driven by a Moyno pump) made up of high-pressure hose and 1-inch pipe then took the grout from the plant t[illegible]gh the working gallery back to the sump. The grout was screened prior [illegible]ering the sump.

Grout mixes were composed of varying amounts of portland cement and water with bentonite. Water/cement ratios varied from 5 cubic feet of water: 1 bag (94 lb.) of cement to 1:1, with the average mix being placed in approximately 3.5:1. The contract provided for bentonite to be added at a rate of 4% (by weight) of the cement in the mix. However, experimentation

showed that 2% bentonite gave a better strength with nearly the same low shrinkage ratio as the 4% mix. The 2% bentonite mix with portland cement and water was used throughout the job.

Grout was carried from the circulating line to the hole by means of a T-shaped header made up of iron pipe, valves, gauges, and high pressure hose. The crossbar of the T was hooked into the circulating line with a valve to control the exhaust side of the T and to maintain pressure in the supply line at as high a level as required to grout. The lower part of the T was hooked into the hole by means of a high pressure hose and a packer. Grout from the supply line went through a cutoff valve and a gauge (equipped with a gauge-saver) before entering the hose and packer. Pressure in the hole was maintained by the valve and monitored by the gauge during operations. Mechanical and pneumatic packers were used to connect the grout line to the hole. Mechanical packers were used in all holes and all zones, while pneumatic packers were used in the first zone of hole 8+27 and all holes with higher numbers (east from there to the South Carolina abutment) for reasons stated below.

After the extensive leaks described below in this section were encountered, it was decided to grout the rest of the holes (holes from Station 8+85 to the South Carolina end of the gallery) in two steps. A pneumatic packer would be set just below the concrete/rock interface and the hole grouted at maximum pressure starting with a thin 5:1 mix. After the hole refused (took 1 foot or less liquid in 10 minutes), the pneumatic packer was withdrawn and a mechanical packer fixed in the grout nipple at the collar. The hole was then grouted a second time starting with a 3:1 mix. In this way, it was hoped we could get the curtain grouting done in the normal way, and then grout any voids in the concrete with a thicker mix since consolidation grouting was what we desired here.

Pressures used during grouting were held to a maximum total pressure of 40 psi in the first zone, 80 psi in the second, and 120 psi in the third. Gauge pressure was calculated as maximum total pressure minus the weight of the full column of the grout being used at the time. The pressure at the header gauge was thus varied with the mix being used at the time. For example, in the third zone, a mix of 5:1 grout would be pressured an additional 58 PSI, while 1:1 grout in the same zone would be pressured an additional 33 psi. When grouting in the first zone was done within 100 feet of any uplift cell, that well was pressurized to 15 psi. In most cases, this seemed to prevent grout infiltration into the uplift cells. Where grout was seen in uplift cell lines, the line, and hopefully the cell, was washed out with water immediately after grouting.

Pertinent data on grouting operations beneath the concrete dam is summarized on Plates 51 and 52. Hole numbers used on the charts and in the text of this report were assigned on the basis of stationing along the dam axis (Station 0+00 is against the gallery wall in Monolith 1). The true location of a grout hole is usually within .5 feet along station of this index in the upstream gutter. However, where the working gallery runs perpendicular to the axis (making all holes essentially at the same station) the holes are numbered consecutively until one hole corresponds with its station. Then stations and numbers are again essentially identical.

During curtain grouting operations, we had several problems with leaks. We noted communication of grout holes with construction joints (the joint between Blocks 2 and 3 and the joint between Monoliths 4 and 5), drain hole nipples prior to drilling of the drains (several places throughout the dam), porous areas in the gallery floor (in Blocks 1, 26, and 27), and outside the dam.

Three of these leaks may be of importance later. During pressure testing, hole 0+10 in the Georgia abutment fan communicated with the area outside the dam where fill was being placed. Fill placement was discontinued to allow excavation to and investigation of the leak. Concrete was removed for a few inches into the leak to reveal a pipe which was carrying water into the fill. It is probable that the grout hole, some 26 feet from the end of the dam was connected to the outside through a lift line (possibly through "honeycombed" concrete or a rock pocket) to a bracing pipe which had held up a form during concrete placement. Water under pressure had infiltrated through this connection, broken through an inch or less of concrete over the pipe opening, and penetrated the fill, collecting on the surface. The leak was repaired by fully exposing the pipe, attaching a valve thereto, and grouting the connection with 1:1 grout.

Hole 16+93 near the Monolith 28/29 construction joint also showed wide communication. When zone 1 of this hole was drilled, the driller reported a 4"-6" tool drop. Pressure testing of the hole revealed communication with another grout hole and an uplift cell in Block 29. Later testing with dyed water showed that the hole also communicated with a hole in the "crosspiece" of the T-shaped gallery in Monolith 28 and with a drain which ran along the 28/29 stepup face. This hole was grouted with 1:1 grout pigmented with iron oxide powder. An exploratory hole drilled later intersected 4" of the brick-red grout in a natural seam. This hole was drilled in the area where the probable relief joint in the South Carolina abutment terminated. The hole probably intersected the joint, affording connection with another hole which penetrated the joint and with the drain along the concrete-rock interface.

Hole 18+03 also had a connection to an area outside of the dam. During drilling and water testing of this hole, workmen on the earth embankment foundation preparation crew noted water coming from underneath dental concrete near the downstream termination of the construction joint between Monoliths 30 and 31. The location of the outside leaks was noted and monitored during grouting of this hole. The points of leakage quickly became wet when zone 1 was being grouted with the mechanical packer. Although we started with a 5:1 mix, no cement was noted at the leak locations. Only (apparently) clear water came through. The hole probably penetrated a thin vertical joint or seam in the foundation which continued into the earth embankment foundation. The seam was apparently wide enough to allow water to pass, but tended to filter out cement particles from the grout. Two quaternary holes drilled 2.5 feet to either side of hole 18+03 took only enough grout to fill the hole volume.

Two mistakes made during grouting should be mentioned. A hole drilled near the Monolith 7/8 construction joint penetrated the edge of the copper waterstop in that joint. As soon as the core (containing a semicircle of copper sheeting wrapped around a section of anchor rod) was pulled, the hole

was terminated. After we determined what the drill bit had cut, we grouted the hole with a thick, sanded grout mix and drilled the hole 1 foot farther away from the construction joint.

Hole 15+71 ruptured the 14" pipe which cased the inverted plum bob hole. This hole was planned to go 1 foot to the South Carolina side of the IPB pipe. However, when it was drilled, the bit intersected the pipe tangentially, cutting an elliptical piece of metal from the circumference of the pipe. This was not discovered until grouting of the hole began. As soon as the mistake was discovered, grouting was stopped and the pipe was washed out. The hole was repaired by using a large packer to seal off the IPB pipe and filling the grout hole with a thick, sanded grout mix. Hole 15+71 was redrilled approximately one foot away from this filled, partially penetrating grout hole.

The maximum distance over which hole to hole communication was noted was approximately 100 feet, when grout pumped into hole 0+81 exited hole 1+80 and several intermediate holes during third zone grouting. Communication over 60 feet or more was rare and over 20 feet or more uncommon. Communication between grout holes and other structures include the following (where a structure is called simply a "drain", it is an undrilled drain nipple): hole 0+10 and outside the dam (previously described), hole 0+14 and a porous area in the gutter and gallery floor, 0+66 and the gallery floor, 1+00, a drain, and the M2/3 construction joint, 2+18 and the M4/5 construction joint, 3+39, a drain, and an angled pipe coming up into the gutter on the stairway from a constructed drain between Blocks 7 and 8 (this drain was grouted up), 3+52, the roof of the stairwell, and the upstream face drain closest to the hole, 5+11 and an uplift cell in Block 10, 6+01 with the two closest drains, 8+30 and a drain, 8+40 and a drain, 8+50 and a drain, 9+45 and a drain, 9+65 and a drain, 9+69 and a drain, 15+61 and a drain, 15+71 and an IPB hole (previously described), 16+32 and a drain, 16+80 and an uplift cell in Block 29, 16+93, a constructed drain, and an uplift cell in Monolith 29 (previously described), and 18+03 and outside the dam (previously described).

Table 10 summarizes actual and planned quantities used during concrete dam grouting. It is obvious from this table that less drilling was used than planned, less grout was used than expected, a lower grout placement/drilling ratio than expected was achieved, and the average grout mix was placed at less than a 3:1 w/c ratio. In spite of these things, the integrity of the grout curtain seems to very good. Reasons for this evaluation are the following: exploratory drill holes along the curtain showed no ungrouted open joints, uplift arrays in Monoliths 10 and 23 showed lower pressure downstream behind the grout and drainage curtains during times that water behind the dam was appreciably higher than the Clarks Hill Lake level, split spaced holes usually took little or no grout, and an investigation to determine the reason for the anomalous leakage described above seemed to indicate that the curtain was doing its job. Possible reasons for the grout underrun are: (1) the majority of the weathered, open jointed rock was removed, particularly in the abutments, (2) rock in the spillway and intake sections was not deeply weathered and the top 5-10 feet of rock was removed (5-7 feet below design grade) due to existing (probable) relief joints, and (3) the metadacite exhibits fewer joints than the higher grade gneisses and schists of the Clarks Hill, Hartwell, and West Point Dams

TABLE 10

RICHARD B. RUSSELL

CONCRETE DAM GROUTING

AS OF 1/1/82

(a) ESTIMATED QUANTITIES

GROUT HOLE DRILLING	-	46,500 LF
PLACING GROUT	-	12,000 CF
GROUT RATIO	-	12,000/46,500 = 0.258 CF/LF

(b) ACTUAL QUANTITIES

GROUT HOLE DRILLING	-	38,316.5 LF
REDRILL	-	8,030 LF
GROUT HOLE DRILLING (ROCK)	-	30,286.5 LF
PLACING GROUT	-	1269 CF
GROUT RATIO	-	1269/30,286.5 = 0.041 CF/LF

DRILLING IS 82% OF BID ESTIMATED QUANTITIES (65% BASED ON ROCK).

GROUTING IS 11% OF BID ESTIMATED QUANTITIES.

(c) PLACEMENT

HOLE SIZE	-	1 1/2 INCH OR 0.125 FEET IN DIAMETER
VOLUME OF HOLE	-	$\pi \times (0.125/2)^2 \times 30,286.5 = 372.5$ CF
AVERAGE MIX USED	-	3.5:1 (WATER/CEMEMT RATIO)

1269 CF SOLIDS (5076 CF MIX) PLACED IN 372.5 CF OF HOLE
OR 372.5 CF SOLIDS WERE USED TO FILL THE HOLE AND
896.5 CF SOLIDS INJECTED INTO THE ROCK.

29.3% OF THE GROUT WAS USED TO FILL THE HOLES.
70.7% OF THE GROUT WAS INJECTED INTO THE ROCK.

upon which the estimate was based. If the spillway foundation had been excavated to original design grade, the open relief joints would have taken a considerable amount of grout and the probable communication and high takes would have required more split spaced holes, increasing the amount of drilling also. Similar considerations apply to the South Carolina abutment. The specifications state that "between Station 9+95.5 and Station 18+57.5, 2-1/2-foot spacing is anticipated within the first zone". The actual condition of the foundation rock was fresh, hard, and unweathered with most joints and seams tight. These tight fractures in the rock would take thin grout mixes, probably filtering out cement particles near the curtain as observed in the outside communication with hole 18+03. If this conjecture is true, the grout remaining in openings near the curtain holes should be more substantial than the 3.5:1 average mix would indicate. In any case, the final verdict on the grout curtain will not be in until filling of the reservoir and time reveals the true results. Additional analysis of curtain grouting is given in Table 9, pages 48 and 49.

10.3-CD Drainage Provisions: Contract Specifications provided for 2-7/8 inch diameter (minimum) holes to be drilled at a 15 degree angle from vertical in a direction downstream and perpendicular to the dam axis. These holes were collared in the downstream corner of the working gallery in small tributary gutters running perpendicular to and ending in the upstream gutters. They were drilled with rotary equipment and diamond bits of NX size to a (vertical) depth of 75 feet on 10-foot centers. Water from these holes runs through the tributary gutters to the upstream gutters and down to sumps in Monoliths 9 and 19 where it is pumped into Clarks Hill Lake.

10.4-CD Dental Concrete and "Broom" Grouting: No dental concrete was used on the Russell dam foundation. The foundation surface over most of the area was irregular and had considerable relief, but the low areas were usually large enough to allow for a "plug" pour prior to mass placement. Where there were narrow depressions in the foundation, concrete inspectors would call for a bucket of concrete with smaller (usually 1-1/2-inch) aggregate and more cementitious material. This bucket would be placed in the depression after application of the basal grout mix.

The basal grout mix used for "broom" grouting on the Russell foundation was composed of 2187 lbs. sand, 629 lbs. cement, 164 lbs. fly ash, 470 lbs. water and 5-1/2 oz. air entraining admixture (per cubic yd.). It was swung into the placement area by gantry cranes (usually) 1-2 yd. at a time. This mix was placed on the wet foundation rock just ahead of concrete placement and "broomed" into any existing cracks and crevices.

10.5-CD Other Treatment and Problems: Problems with flaws in the foundation rock, water, etc., have been dealt with in this report under sections covering the part of the work they affected. The only treatment which has not been covered is the adjustment made due to the abrupt irregularities in foundation topography. Throughout the job, where a stepup face existed, the face was removed if the angle of the face was such as to prevent good vibration of the mass concrete. Also any sharp corners were removed to avoid focal points for stress cracks in the concrete. An additional adjustment made during construction (after recommendation by the District Structural Section) was to use 3-inch, 2000 lb. mass concrete on the foundation and to limit lift height to 3 feet, rather than the usual

7.5, where foundations were very irregular. This was done on the South Carolina abutment monoliths and some in the spillway and intake areas. Prior to this change, a 6-inch, 2000 lb. mass concrete had been used.

10.0 FOUNDATION TREATMENT - EMBANKMENTS

10.1-EE Foundation Grouting: A grout curtain was installed beneath the earth embankments to prevent or slow down seepage; to increase the length of the flow lines through the foundation; and to attempt to fill all open or partially open joints and other openings within the foundation rock. Plates 84 and 85 show the grouting layout in plan view, while Plates 86 thru 92 are profiles of the grout curtain for selected areas. Refer to Table 11 for a summary of earth embankment grouting. In addition to the curtain grouting described in this section, Section 7.1.4-EE describes the consolidation grouting program which was utilized in the diversion channel to consolidate the fault zone encountered there.

Initial grout plans called for a single-line grout curtain parallel to and 10 feet upstream of the dam centerline with grout holes angled down-station at approximately 20 degrees from the vertical. Except between Station 14+20 and Station 18+20, it was anticipated that the grout curtain would extend to a depth of 90 feet (Zone 3) for the primary (set 1 on 20-foot centers) holes; 50 feet (Zone 2) for the secondary (set 2, split-spaced on primaries) holes; and 20 feet (Zone 1) for the tertiary (set 3, split-spaced on secondaries) holes. Between Station 14+20 and Station 18+20 (the diversion channel crossing) a fourth zone was added for a total depth of 125 feet for the primary (20-foot centers) holes. Zone 3 (90 feet) holes were on 10-foot spacing and Zones 1 and 2 were on 5-foot spacing. Additionally, the 20 degrees west inclination of the grout holes was waived in lieu of a transition fan to a 20 degrees easterly pattern. This change was necessary for advancing the grout holes into the opposing slope on the Georgia east side of the diversion channel. As the curtain progressed from west to east, the orientation reverted to 20 degrees west once grouting from a relatively "flat" surface resumed (Station 16+20 approximately), see Plate 89.

10.1.1-EE Multiple Grout Lines: Multiple grout lines were used where grout takes for the "A" line so indicated. The second grout line (the "C" line) was placed on the dam centerline and the "B" line was located halfway between the "A" and "C" lines (reference Plates 84 and 85). Holes of adjacent lines were staggered relative to hole stationing on nearby lines. In the event more than three lines were required (as shown on Plate 84, the additional lines were located upstream of the "A" line. In no instances were grout lines permitted downstream of the dam centerline except for the consolidation grouting of the diversion channel fault zone (see Section 7.1.4-EE of this report and Plate 108 for details on consolidation grouting).

10.1.2-EE Grouting Procedures: For a given section of grout line, all primary holes were drilled, pressure washed, tested to Zone 1 depths, and grouted. If grout takes indicated the need for split-spacing, secondary holes were drilled and grouted as for the primary holes. Any additional split-spaced holes were then added and the process repeated until Zone 1 was accepted as being completely grouted. After Zone 1 was accepted, the process was repeated for Zones 2, 3, and 4 in order (i.e., all primary holes

TABLE 11

RICHARD B. RUSSELL

EARTH EMBANKMENT GROUTING

AS OF 4/1/82

(a) ESTIMATED QUANTITIES

GROUT HOLE DRILLING	-	40,000 LF
PLACING GROUT	-	5,000 CF
GROUT RATIO	-	5,000/40,000 = 0.125 CF/LF

(b) ACTUAL QUANTITIES

GROUT HOLE DRILLING	-	81,059.9 LF
REDRILL	-	27,993.5 LF
GROUT HOLE DRILLING (ROCK)	-	53,066.4 LF
PLACING GROUT	-	10,584.2 CF
GROUT RATIO	-	10,584.2/53,066.4 = 0.199 CF/LF

DRILLING IS 202.6% OF BID ESTIMATED QUANTITIES (133% BASED ON ROCK).

GROUTING IS 211.6% OF BID ESTIMATED QUANTITIES.

(c) PLACEMENT

HOLE SIZE	-	2 1/2 INCH OR 0.2083 FEET IN DIAMETER
VOLUME OF HOLE	-	$\pi \times (0.2083/2)^2 \times 53,066.4 = 1808.4$ CF
AVERAGE MIX USED	-	2.5:1 (WATER/CEMENT RATIO)

10,584.2 CF SOLIDS (31,752.5 CF MIX) PLACED IN 1808.4 CF OF HOLE OR 1808.4 CF OF SOLIDS WERE USED TO FILL THE HOLE AND 8775.8 CF OF SOLIDS INJECTED INTO THE ROCK.

17.1% OF THE GROUT WAS USED TO FILL THE HOLES.
82.9% OF THE GROUT WAS INJECTED INTO THE ROCK.

within a section of the grout curtain were grouted followed by secondaries, tertiaries, and quaternaries; likewise, Zone 1 was completely grouted before Zone 2, and Zone 2 was grouted before Zone 3.

Grouting always proceeded from the topographic low point to the high point within a section. No drilling nor washing was permitted within 200 feet of a previously grouted hole until after a waiting period of at least 24 hours had elapsed since injection of grout. This requirement was later relaxed to 100 feet without ill effect.

Grout hole drilling was performed with air-track drills using air and water as the drilling fluid. The hole was 2 inches in diameter and was drilled through a 3-inch diameter steel pipe which was seated and grouted into the rock. Four-inch casings were used where grouting through overburden was required. These casings were installed through oversized holes augered to refusal. This proved unsatisfactory as communication was noted when the first zone was drilled for grouting. Later holes in these areas were drilled for a short distance into rock, then stage grouted with a 1:1 mix. This usually prevented communication through the overburden, but it is likely that there was some erosion and possibly some hydrofracturing at the overburden/rock interface during drilling of this first stage. A procedure where casings are drilled through overburden under low pressure and seated in the rock in one step should be used in future contracts. The casings were cut off at the ground surface prior to fill placement.

After drilling and immediately prior to grouting, all grout holes were washed with air and water injected through a pipe which was lowered to the bottom of the hole until the return water was clear and contained no cuttings or joint filling material. After washing, the holes were pressure-tested with water at pressures not to exceed 1 psi per foot of hole depth until all return water was clear. For those holes in which the maximum allowable pressure could not be obtained, the holes were washed for 5 minutes with the pumps operating at maximum capacity.

Once all the holes had been washed and pressure-tested within the section to be grouted, grouting operat ns began. Grout was injected through a packer which was set in . -inch diameter steel pipe nipple. Pressures going into the grout hole were controlled at the grout header (at the packer, see Plate 112 for grout header arrangement) with grout being circulated continuously. This arrangement prevented the solids from s tling out of suspension and ensured a homogenous grout mix. Likewise, . e accurate control of the down-hole pressures was obtained.

Grout mixes ranged from 0.6:1 water to cement ratio (by volume) to 5.0:1 water to cement ratio with 2% bentonite (.02 x cement wt.). Normally, grouting began with the thinnest mix (5:1) and was thickened only if grout takes indicated the need for such thicker mixes, so as to obtain refusal. On some grout holes with abnormally large water takes during washing and testing, a 3:1 grout mix was called for initially rather than the 5:1 mix which was the "normal" initial mix.

Refusal criteria for a given hole was considered met if less than one cubic foot of grout at the maximum allowable header pressure could be pumped into the hole in a ten minute time period. Upon obtaining refusal for a

zone, the grout was allowed to achieve its initial set and the hole was then washed out (except when takes were higher than normal) so as to preclude the requirement for redrilling that particular zone. If the hole was complete after reaching refusal, washing of the hole was not performed, but rather the hole was backfilled completely with grout and allowed to set. Upon completion of a hole, the steel pipe nipple was either removed or cut off even with the ground surface. For a graphic description of general grouting procedures refer to Plates 110 and 111.

10.1.3-EE <u>Effectiveness of Curtain Grouting</u>: A visual inspection of the downstream embankment toe areas could be indicative of the success of the grout curtain. The absence of unusual seepage on the downstream sides of the embankments can be attributed at least in part to a comprehensive grouting program. More information regarding the grout curtain's effectiveness will be made available when the Embankment Criteria and Performance Report is completed.

10.2-EE <u>Foundation Compaction and Consolidation</u>: Consolidation of the hanging wall of the diversion channel fault was achieved by grouting. This grouting operation is described in Section 7.1.4-EE of this report.

10.3-EE <u>Dental Concrete</u>: Dental concrete was used to fill narrow depressions formed by blasting, ripping, or dental excavation, and against vertical or overhanging rock ledges. Dental excavation was performed on all deeply weathered or extremely shattered rock (primarily vertical mafic dikes) using hand tools and/or loader-backhoes. A Cat 245 hydraulic excavator was used in the diversion channel for dental excavation of the fault. The depth of excavation criteria used was generally to excavate to a depth equal to two times the width of the excavation. The concrete used was a 2500 psi ready-mix which was vibrated and wood trowel finished. A low-slump mix was used for the fillets and a slightly higher slump was specified for dike excavations. Forms were not normally required, but a toe board was used for fillets over about three feet in height. Section 7.5-EE of this report describes a large (244 cubic yards of concrete) fillet which was required in the South Carolina embankment cutoff trench. Contract quantities for both dental excavation and dental concrete were entirely too low and resulted in large overruns (see Appendix G, Variations In Estimated Quantities). For further information about locations that required dental concrete placement refer to the Embankment Criteria and Performance Report.

10.4-EE <u>Fibrous Shotcrete Treatment</u>: Upon completion of all other forms of foundation treatment, a fibrous reinforced shotcrete was applied to the surface of all firm and fractured sound rock. The intent was to minimize the possibility of the highly erodible impervious core material from being transported into openings in the foundation rock. The material was a dry-mix of sand, cement, and steel fibers applied pneumatically with water added at the nozzle. Specifications called for a minimum of 3 inches of shotcrete; any test revealing less would require an additional 3 inches at the contractor's expense. There was no specified restriction on maximum thickness. In generally flat areas, the measured thickness was normally 3-1/4 to 3-1/2 inches (contractor used 3-inch long nails as thickness guides). However, in irregular surface areas with abrupt changes, thicknesses on the order of 6 to 8 inches were not uncommon in one spot in order to obtain 3 inches on an adjacent higher spot. Consequently, many abrupt details were eliminated through the use of shotcrete. Many of these areas would have

needed slush-grouting or mortar treatment if shotcrete had not been applied. Test beams were made and cured in the field. Further information about areas that were shotcreted will be included in the Embankment Criteria and Performance Report.

Testing of the shotcrete product was specified to be through the use of panels, beams, and cores. Only the cores sampled the in-place shotcrete. It was determined after a few days that the panels were of very little benefit on a daily basis once a nozzle operator had been approved. It was more desirable to obtain core samples to monitor thickness and consistency. Production rates were initially only 1 to 2 cores per week. Therefore, daily panels were discontinued and daily cores substituted. Of the 116 cores drilled, only 1 revealed less than the required 3" (2" core recovered, this area re-shotcreted).

Laboratory testing of the shotcrete was performed at the South Atlantic Division Laboratory. Shotcrete beams and cores were selected for testing after field curing. Normally, ages of 7, 14, and 28 days were selected, although other ages were occasionally tested. Beams were tested in flexure and in unconfined compression. Tables showing shotcrete test results are contained in Appendices D thru F.

10.0-PH FOUNDATION TREATMENT - POWERHOUSE

10.1-PH Powerhouse Foundation Grouting: Unlike other grout programs at the Russell Project, the powerhouse foundation was grouted for the purpose of foundation consolidation rather than for control of subsurface flows. This being the case, grouting was performed in only one stage with all holes being the same depth. The distribution and locations of the grout holes are shown on Plate 125 and Table 12 lists pertinent data.

The physical location of the grouting was downstream of the service bays and between the draft tubes of each unit. After rock excavation for the unwatering trench, these areas formed "noses" or protrusions of rock which were eventually encased by concrete. The rationale behind leaving these protrusions rather than excavating them and backfilling with concrete was based upon an engineering economics analysis which concluded that it would be more cost effective to leave the rock and grout rather than remove the rock and replace it with concrete. The rationale for grouting the rock was based upon the fact that the rock, rather than being a homogeneous mass, consisted of a finite number of discrete joint blocks with varying amounts of point contact, and since these areas would be subject to vibration from the generator units it was deemed advisable to consolidate them so that their behavior would approach that of a homogeneous mass.

To achieve this homogeneity, it was determined that a portland-cement based, non-shrink, non-expansive grout would be satisfactory from the standpoint of both economy and effectiveness. The contract specified that the grout would consist only of water, Type I portland cement, and unpolished aluminum powder and that the proportions would be designated by the Contracting Officer. This wording of the specifications proved to be very fortuitous as it allowed project personnel considerable leeway in adapting the grout mix to the conditions encountered.

Observations during grouting operations together with intimate knowledge of the geologic characteristics of the project site leads to the conclusion

TABLE 12

TABLE OF GROUT DATA FOR POWERHOUSE

No. of Holes	177 Holes (each 30.0 feet deep)
Linear Footage	5310 feet
Total Grout Take	374.72 ft.3
Lowest Take/Foot	0.0 ft.3
Highest Take/Foot	1.23 ft.3
Mean Take/Foot	0.0705 ft.3
Standard Deviation	0.1492

Total Grout Take as Percentage of Government Estimate - [illegible]

that the grout program objectives have been fully met. The success of the program was largely due to the manner in which the specifications were written (see preceeding paragraph for summary) an it is suggested that careful consideration be given to the idea of writing future grout specifications in a similar fashion as opposed to specifying X number of bags of a specified mix before being allowed to switch to a new grout mix. It is felt that design objectives can often be met a lower cost and in less time if the field personnel are provided with flexible and non-confining specifications.

10.2-PH <u>Foundation Drainage</u>: Foundation drains in the powerhouse structure are designed to intercept and divert any subsurface waters whose hydraulic potential is great enough to pose hydrostatic uplift problems. The foundation drainage system is composed of three main types of elements. The first of these is the system of relief wells drilled into rock underneath the service bay and draft tube floor slabs. Plate 126 depicts the location of these drains while Plate 129 depicts typical profiles. These drains are 3-inch diameter holes drilled forty feet into rock and are designed to intercept, at depth, any groundwater which may be influenced by either Clarks Hill or Richard B. Russell impoundments.

The second type of drainage element utilized in the powerhouse is a system of weep holes which extend through the draft tube floor slabs and are connected to collector boxes which are installed at the concrete/rock interface. The collector boxes are half-round corrugated PVC, concave up, which were installed over open joints in the foundation rock. To prevent dislocation during concrete placement, these boxes were pinned to the rock with short lengths of re-bar and sealed against intrusion of concrete by cement mortar. Plates 126 and 129 show the locations of the weep holes and typical sections of the collector box system.

The third drainage element is located in the erection bay substructure wall and the tailrace hang-on wall. Weep holes through the tailrace wall and a lateral drain along the erection bay wall were designed to limit the backfill saturation level to three feet above tailwater for normal conditions.

10.3-PH <u>French Drains</u>: In those areas of the powerhouse foundation where in-flows of water occurred, french drains were installed prior to placement of concrete. The riser pipes were extended as high as necessary to overcome hydrostatic head and when concrete had been placed to this level, the drains were grouted to refusal (when only one riser was utilized) or until grout return through the second riser (in the case of extensive linear in-flow zones) was achieved. The majority of the french drains were installed in the bottom of the unwatering trench where sub-horizontal relief joints day-lighted in the walls of the trench. In those cases where only a vertical large-diameter pipe was sufficient to isolate the in-flow, a sump pump was used to control the in-flowing water until concrete had been placed above the piezometric surface. Once this was accomplished, a small diameter grout pipe was installed in the drain, the pump was removed and the large pipe was backfilled with a clean crushed stone and grout was pumped into the drain until all water had been displaced and grout return at the surface achieved.

10.4-PH <u>Foundation Preparation</u>: Pry bars, wedges, jack hammers, and water jets were used in preparing the foundation for concrete. All loose and drummy rock, rock underlain by clay seams, overhanging rock ledges, and excessive staining and mineral deposits were removed under the direction and supervision of the Project Geologists who had final authority regarding the suitability of the foundation. Most areas of the foundation required extensive rock removal due to the intensive jointing, the presence of numerous shear zones, and the effects of blasting during initial excavation. Once the foundation satisfied the Project Geologists, the areas were photographed (see Appendix K) and mapped and the foundation was approved for rebar and concrete. Any excessive delays between foundation approval and concrete placement required a second inspection and approval by the Geologists immediately prior to placement.

10.5-PH <u>Foundation Mapping and Approval</u>: The entire powerhouse foundation area was mapped by the Project Geologists. Field mapping was at the scale of 1:60 with the final map (Plates 120-123) at a 1:240 scale. Due to the nature of the construction sequence, the largest area available for mapping at any one time was the service bay areas and so it was considered that the use of a plane table and alidade was unnecessary. Instead, each area was laid out in a 5-foot grid and geologic features were located relative to the intersections of the grid. Among those features plotted on the maps were rock type, contacts, discontinuities, mineralization, and any other features which were deemed to be of importance in the event of any future foundation problems. Before mapping commenced, the foundation was examined by a Geologist and only after the foundation was considered to be ready for concrete was it mapped. Foundation approval and mapping was conducted from December 1981 through August 1983 when the last of the rock was finally covered by concrete. In addition, mapping of the tailrace slope was accomplished in 1984, immediately prior to inundation.

11.0 FOUNDATION INSTRUMENTATION - CONCRETE DAM

11.1 Concrete Dam Structures: The three types of structures in the concrete dam are: non-overflow, power-intake, and a gated spillway section. Bearing pressure on the foundation is less than 15 tons per square foot. Sliding or shearing stress on the structures is less than 100 psi. These loads are well within the capacity of the foundation (refer to Section 3.5 for more details).

Plates 53 thru 56 show the design details and locations of instruments that were planned for the dam. Plate 57 shows final locations of instruments. Foundation instrumentation installed in the concrete dam includes: Uplift cells in Monoliths 7, 10, 23, 26, 27, 28, and 29, side by side plumb bobs and inverted plumb bobs in Monoliths 16 (inverted plumb bob is 100 feet into the foundation) and 26 (inverted plumb bob is 85 feet into foundation), and deflectometers (inclinometers) in Monoliths 27 and 28.

11.2-CD Purposes: Instrumentation was implemented on this project in order to maintain data on: (1) hydrostatic uplift on the foundation under several of the concrete dam monoliths, (2) deflection and tilting of individual monoliths, (3) rates of inflow of water into the inspection gallery, and (4) seismic activity.

11.3-CD Uplift Cells: The locations of uplift cells in the foundation were based on a random sample of the various type of structures above the foundation and variations in elevations and conditions of the foundation. The blocks and types of structures that were selected are as follows:

Block 7 - Georgia non-overflow.

Block 10 - Power intake section.

Block 23 - Typical spillway section

Block 26 - End block of spillway

Block 27 - South Carolina - non-overflow.

Blocks 28 and 29 - Variation in rock condition under South Carolina non-overflow.

Uplift cells were located from the upstream edges of the blocks to the downstream edges of the blocks in order to determine the variations in uplift pressure from upstream to downstream. Uplift cells were also located both upstream and downstream of the foundation drains to determine the effectiveness of the drains. The uplift cells were placed on top of rock, 4 feet deep and 10 feet deep in order to monitor the uplift conditions at the contact plane and various levels below the foundation surface.

11.4-CD Plumb Bobs: Plumb bobs were installed in spillway end blocks to measure tilting of the dam under reservoir loads. Inverted plumb bobs were also installed to determine if the pool and load from the concrete dam effected the foundation. The measurement of the plumb bobs are compared

against the alignment surveys to verify the accuracy of the surveys. The data is used to evaluate the safety of the structure.

11.5-CD Inclinometers: Inclinometers were installed in Blocks 28 and 29 for the same purposes as the plumb bobs. Inclinometers are considerably less expensive than the plumb bobs, but are at the same time slightly less accurate.

11.6-CD Seismic Instrumentation: Earthquake instrumentation consists of two strong-motion accelerographs, one Wilmot Seismoscope, and one peak-recording accelerograph.

The strong motion accelerographs are activated by strong locally occurring seismic activity. The data is recorded on 70 MM film stored in a magazine inside the instrument. The unit is battery operated and its triggering mechanism is electrically activated.

11.0-EE INSTRUMENTATION - EMBANKMENTS

Instrumentation prescribed for the earth embankments included open-system type piezometers, gas-operated piezometers, settlement plates, inclinometer / settlement gages, surface reference monuments, and benchmarks. Refer to Plates 113 and 114 for design locations of these monitoring devices. Plate 115 contains installation and functional details.

The as-built locations and descriptions of the instruments actually installed in the project (which differed somewhat from designed locations and descriptions) will be assimilated into the Earth Embankment Criteria and Performance Report.

12.0-CD POSSIBLE FUTURE PROBLEMS - CONCRETE DAM

12.1-CD Conditions That Could Produce Problems:

a. Relief joints in the Georgia abutment - These are described in previous sections. They are clean, irregular, and show good rock to rock contact where exposed, so they were not considered to pose any structural danger to the dam. Any voids should have been filled by grout (communication was evident along these joints during grouting operations). However, any geologist examining the Russell Dam foundation should be aware of their presence. If portions of these extensive planes were incompletely grouted, they could provide avenues for seepage.

b. The Monolith 28/29 step-up face - Block 28 was excavated much deeper than originally designed due to weathered and deteriorated rock. A dike swarm/shear zone with evidence of multiple movement episodes cuts the face. A near horizontal seam with clay and mud filling terminated irregularly in Blocks 28 and 29. Triangular remnants of this seam still exist in the upstream and downstream corners of the Block 29 foundation on the west side of the block. The grouting plan here was altered to cut off flow at the face. However, the area should be monitored for excess seepage.

12.2-CD Recommended Observations: The areas mentioned above should be monitored for seepage. Uplift cells in Monolith 7 should be helpful for observation of seepage in the Georgia abutment, along with the drains downstream of the grout curtain. The foundations in the Monolith 28/29 area can be monitored by uplift cells in both blocks and in Blocks 26 and 27. Drains downstream in these blocks and horizontal drains in the Monolith 29 face can also be observed.

12.3-CD Lessons Learned: Some effort should be made to require the contractor who blasts rock to bear some of the burden of overblasting. In order to do this, the point at which the work is split into different contracts should be changed from that used on this dam. A possible better point might be at 5 feet above design grade. The excavation contract could be stopped at 5 feet above grade and the concrete construction contractor be required to remove the last 5 feet of rock and to prepare the surface and place concrete. This would also have the advantage of allowing for minor changes in design grade between inspection of excavation and bidding on the construction contract. Alternatively, the excavation contractor could be required to do all excavation and foundation preparation, then to place concrete to 5 feet above design grade. Both of these changes would prevent the problem of having a prepared rock surface spall and loosen in the time between the presentation of the two contracts and would minimize claims and other problems which arise when two contractors have to operate in the same area.

Another possible improvement in current Corps practice is the drilling of probe borings as well as core borings. In locations where the top of rock is extremely irregular such as the Russell area, it is difficult to delineate pinnacles of rock and pockets of soil. Until geophysical methods become discriminating enough to do this, it might be economically beneficial to use a self-propelled drill rig such as an air-trac drill to probe to the top of rock. With a small diameter bit, drilling through overburden to the

top of rock only, a high production rate and low unit cost could be maintained. For every foot of NX core hole eliminated, at least 10 feet of air-trac hole could be drilled. This would allow denser coverage, greatly increasing the chances of weathered linear features such as faults being discovered, and preventing material overruns such as that which occurred on Class I excavation on the Richard B. Russell Dam. The usual splitspoon and coring holes could be drilled to determine type, nature, and engineering properties of materials and suitability of a specific site. Then a pattern of probe borings could be made to determine top of rock topography and depth of overburden. Finally, additional core borings could be made to explore low spots and linear features which showed up in the probe boring pattern. Over 400 borings were drilled for Line C along which the Russell Dam was built. This expense could probably have been lowered, or at least more information to guide design could have been gathered for the same price.

12.0-EE POSSIBLE FUTURE PROBLEMS - EMBANKMENTS

12.1-EE General: The potential for future foundation problems which might constitute a threat to the integrity of the embankment appears to be very low. The probability of foundation conditions developing which would create maintenance or aesthetic problems is not so remote.

12.2-EE Quarry Shear Zone: The only foundation area with a high probability of developing into a problem which will require remedial measures in the future is the Dravo-Groves Quarry shear zone. The Dravo-Groves Quarry was operated by the concrete dam contractor as a source of aggregate and other processed stone for the Richard B. Russell Project and is located within the reservoir area near the South Carolina shore. Plate 3 indicates the location of the quarry relative to other features of the project. Investigation by Government Geologists of a zone of sheared and brecciated rock exposed by quarry operations resulted in the hypothesis that this shear zone extends, in the subsurface, through the South Carolina Saddle Dike foundation. The saddle dike was not founded on rock and no grout curtain was installed under the dike.

Evaluation of the geologic investigation resulted in the decision that no foundation improvement would be performed under the construction contracts. This decision was based upon the supposition that if any remedial work were required that it would be more economical to do so in the future than to immediately engage in a large-scale foundation improvement program when the need for such work was not evident. A system of weirs has been installed in the ravines downstream of the saddle dike and flow rates are being monitored regularly by Operations Division. If the flow rates begin to increase over time, Engineering Division is prepared to initiate an exploration and investigation program and to recommend whatever remedial work (if any) is required.

12.0-PH POSSIBLE FUTURE PROBLEMS - POWERHOUSE

The likelihood of any major problems developing in the powerhouse foundation appears to be very remote. The only foreseeable problems would be that some of the relief wells might become clogged for some reason and it is felt that this would not create a serious problem as the entire structure is well anchored with an extensive network of rock bolts. Also, if the

wells do become clogged, it would be a relatively minor operation to either ream the existing wells or to drill new wells.

13.0 REFERENCES CITED

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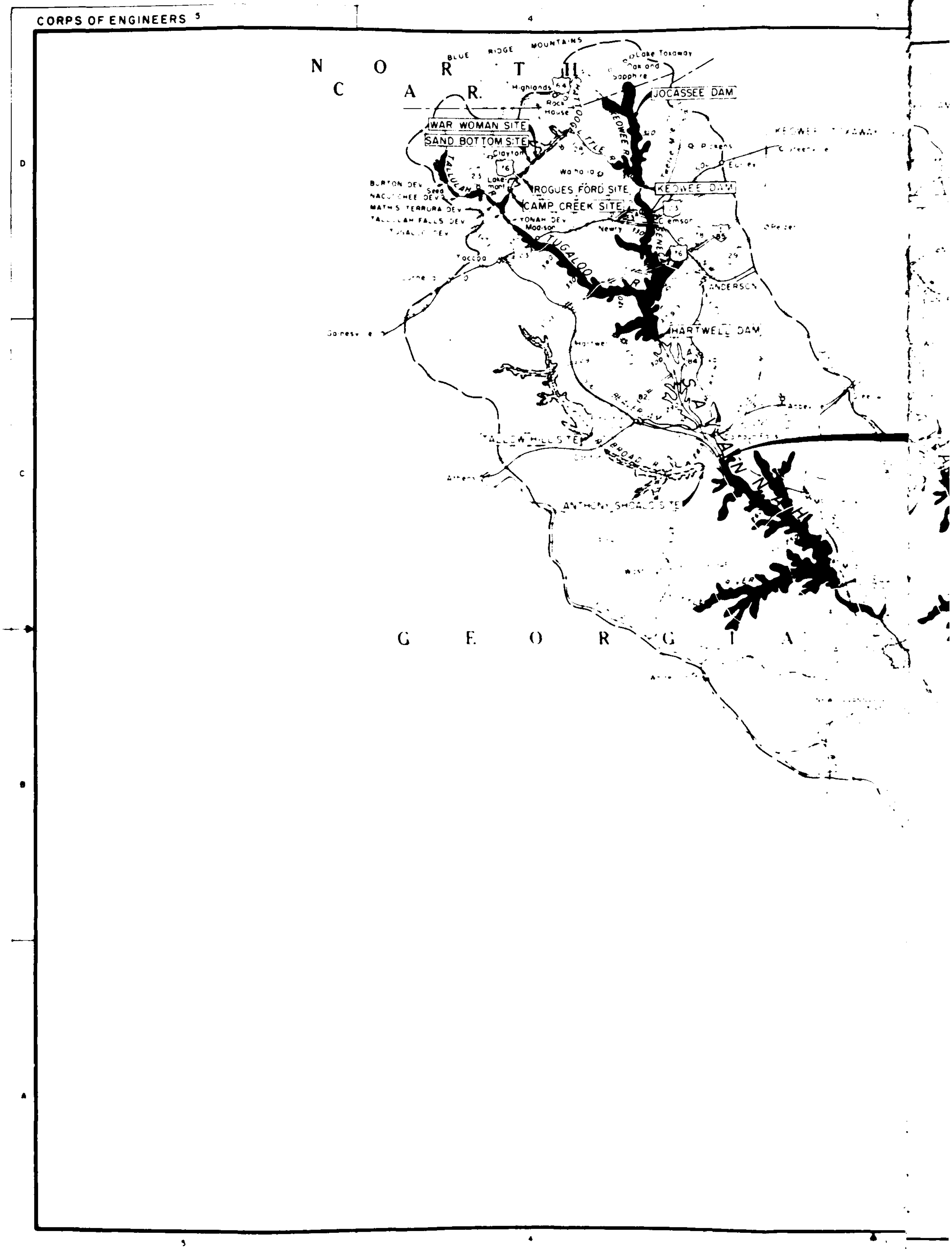
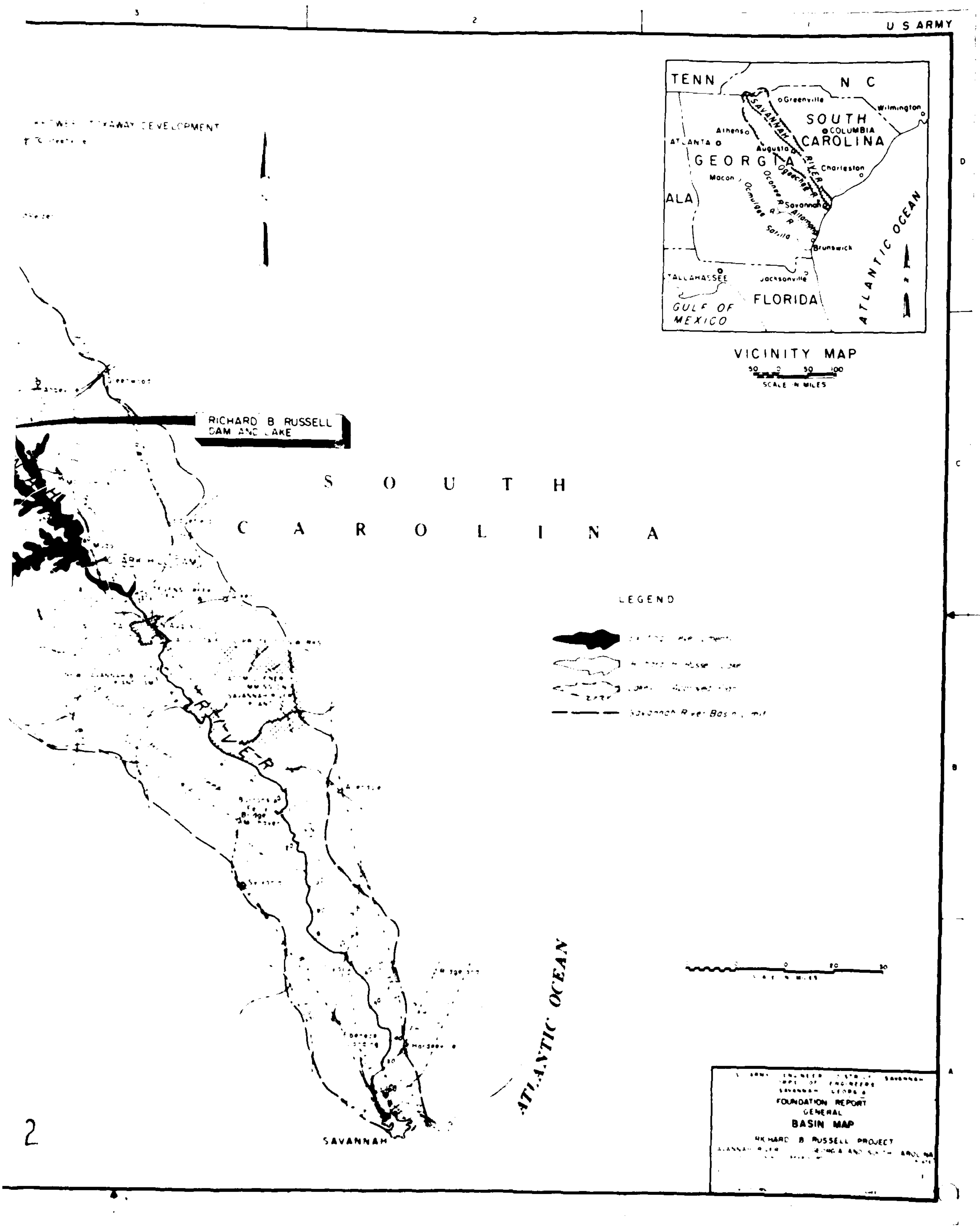
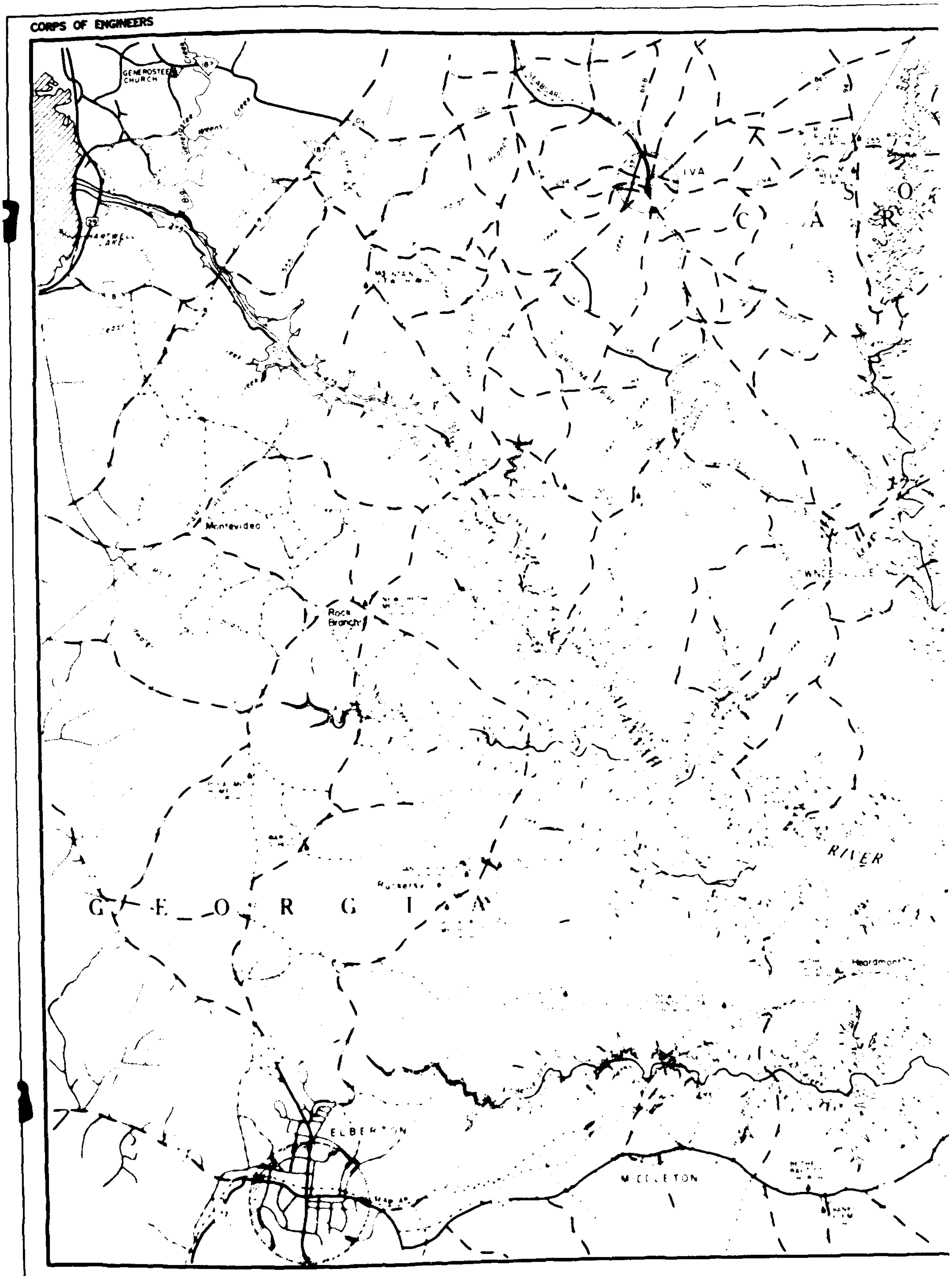
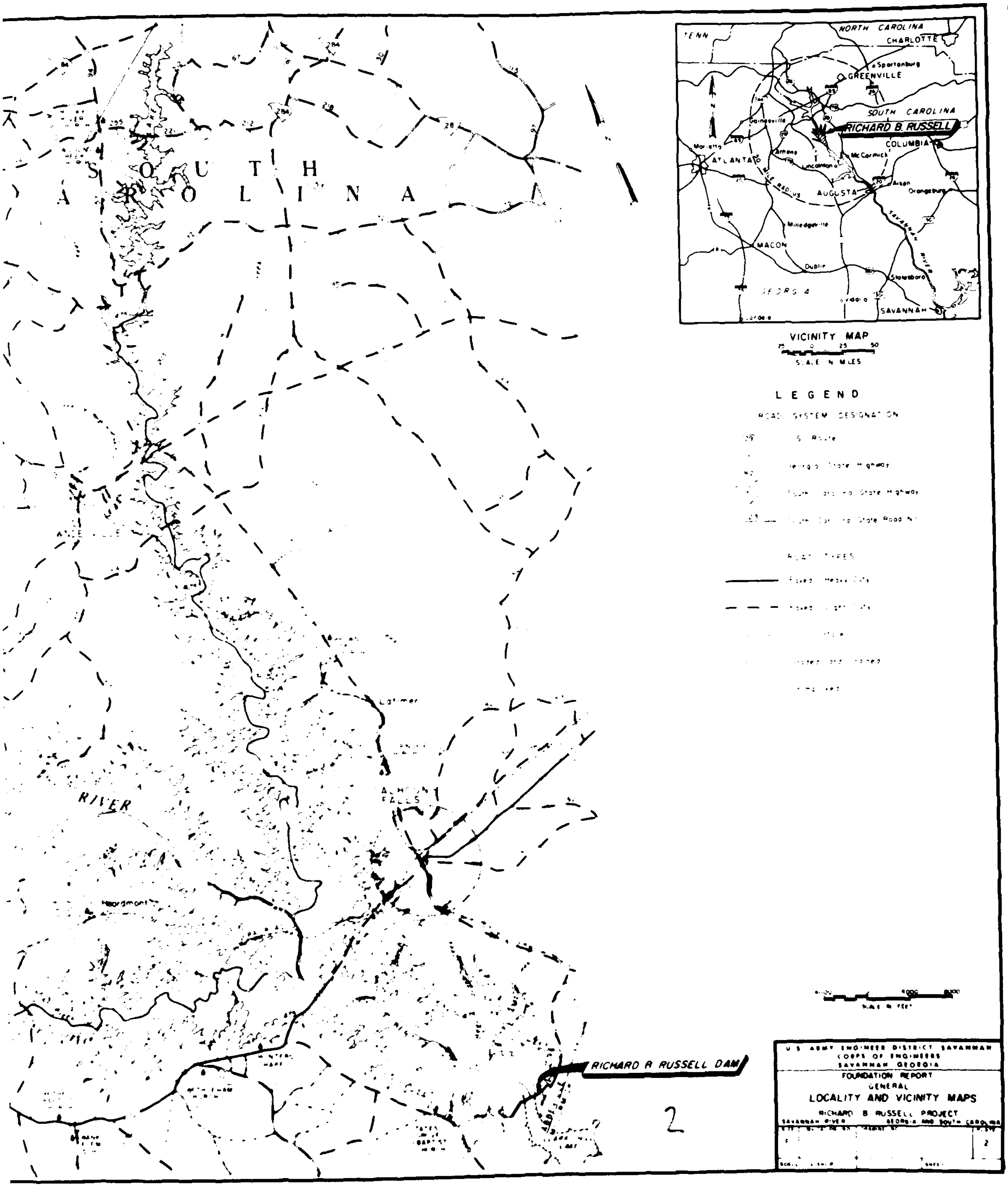
PLATES


CORPS OF ENGINEERS
NORTH CAR.
BLUE RIDGE MOUNTAINS
Highlands
Rock House
Lake Toxaway
JOCASSEE DAM
WAR WOMAN SITE
SAND BOTTOM SITE
Clayton
ROGUES FORD SITE
CAMP CREEK SITE
KEOWEE DAM
BURTON DEV
NACOOCHEE DEV
MATHIS TERRORA DEV
TALLULAH FALLS DEV
YONAH DEV
Madison
Newry
Clemson
Toccoa
TUGALOO R.
ANDERSON
HARTWELL DAM
Gainesville
Athens
ANTHONY SHOALS SITE
GEORGIA


U S ARMY
TENN
N C
Greenville
Wilmington
SOUTH CAROLINA
COLUMBIA
Athens
ATLANTA
Augusta
GEORGIA
SAVANNAH RIVER
Charleston
Macon
Ogeechee R
Oconee R
Ocmulgee R
Altamaha
Savannah
ALA
Satilla
Brunswick
TALLAHASSEE
Jacksonville
FLORIDA
GULF OF MEXICO
ATLANTIC OCEAN
VICINITY MAP
SCALE IN MILES
RICHARD B RUSSELL DAM AND LAKE
SOUTH CAROLINA
RIVER
LEGEND
Savannah River Basin Limit
SAVANNAH
ATLANTIC OCEAN
FOUNDATION REPORT
GENERAL
BASIN MAP
RICHARD B RUSSELL PROJECT


CORPS OF ENGINEERS
GENEROSTEE CHURCH
IVA
Montevideo
Rock Branch
G E O R G I A
C A R O
RIVER
Heardmont
ELBERTON



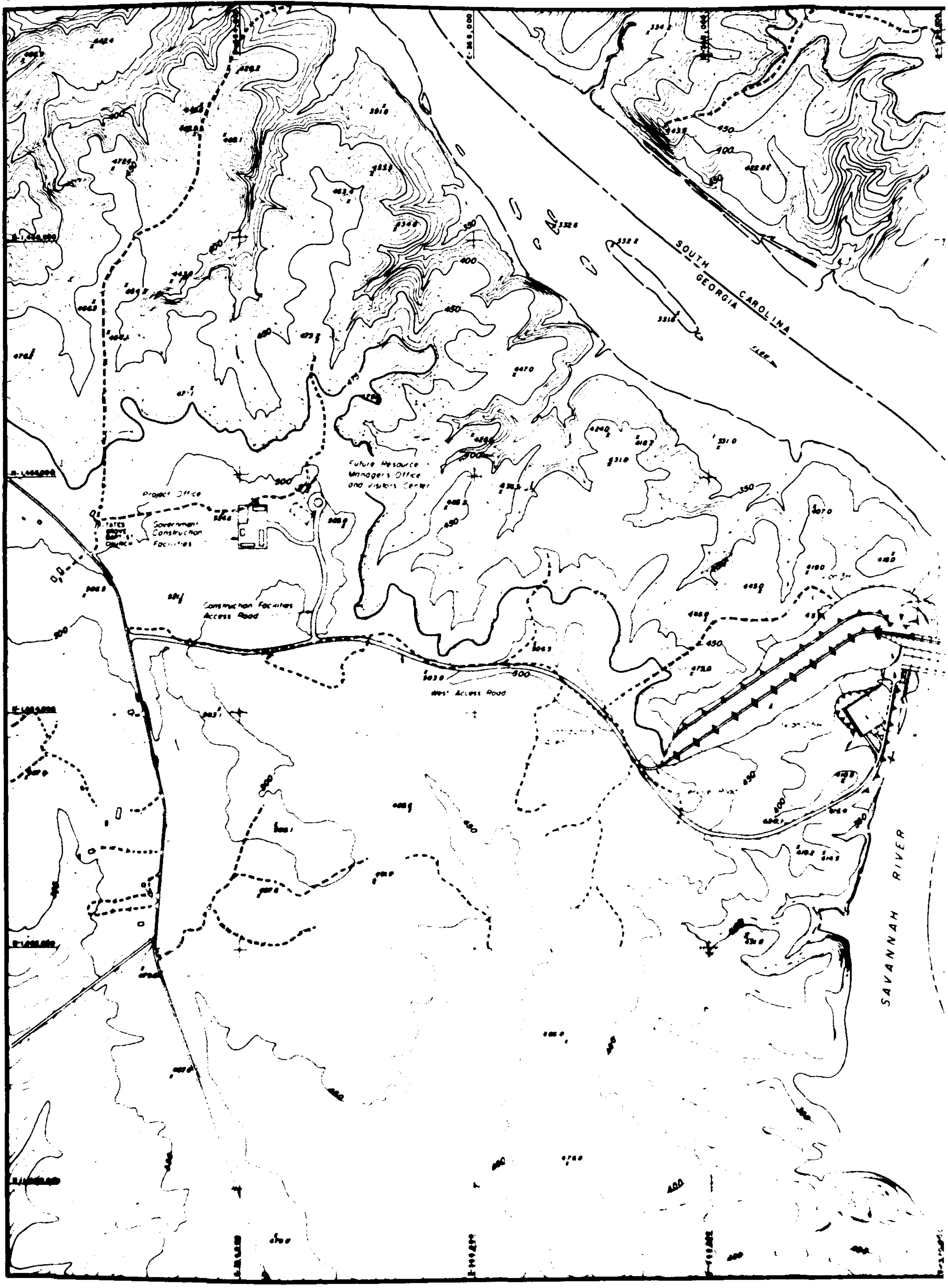
U S ARMY ENGINEER DISTRICT SAVANNAH
CORPS OF ENGINEERS
SAVANNAH GEORGIA

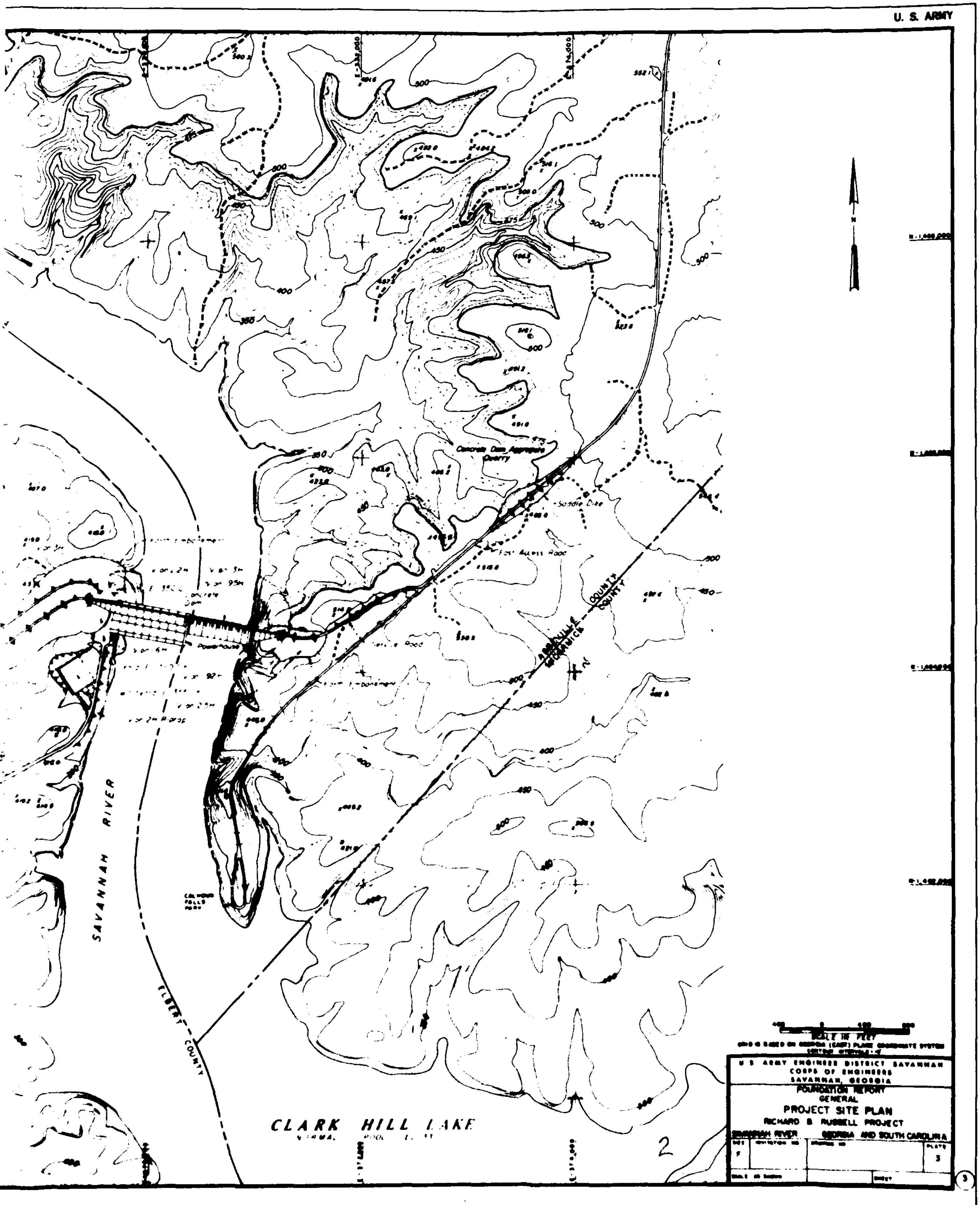
FOUNDATION REPORT
GENERAL
LOCALITY AND VICINITY MAPS
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

2

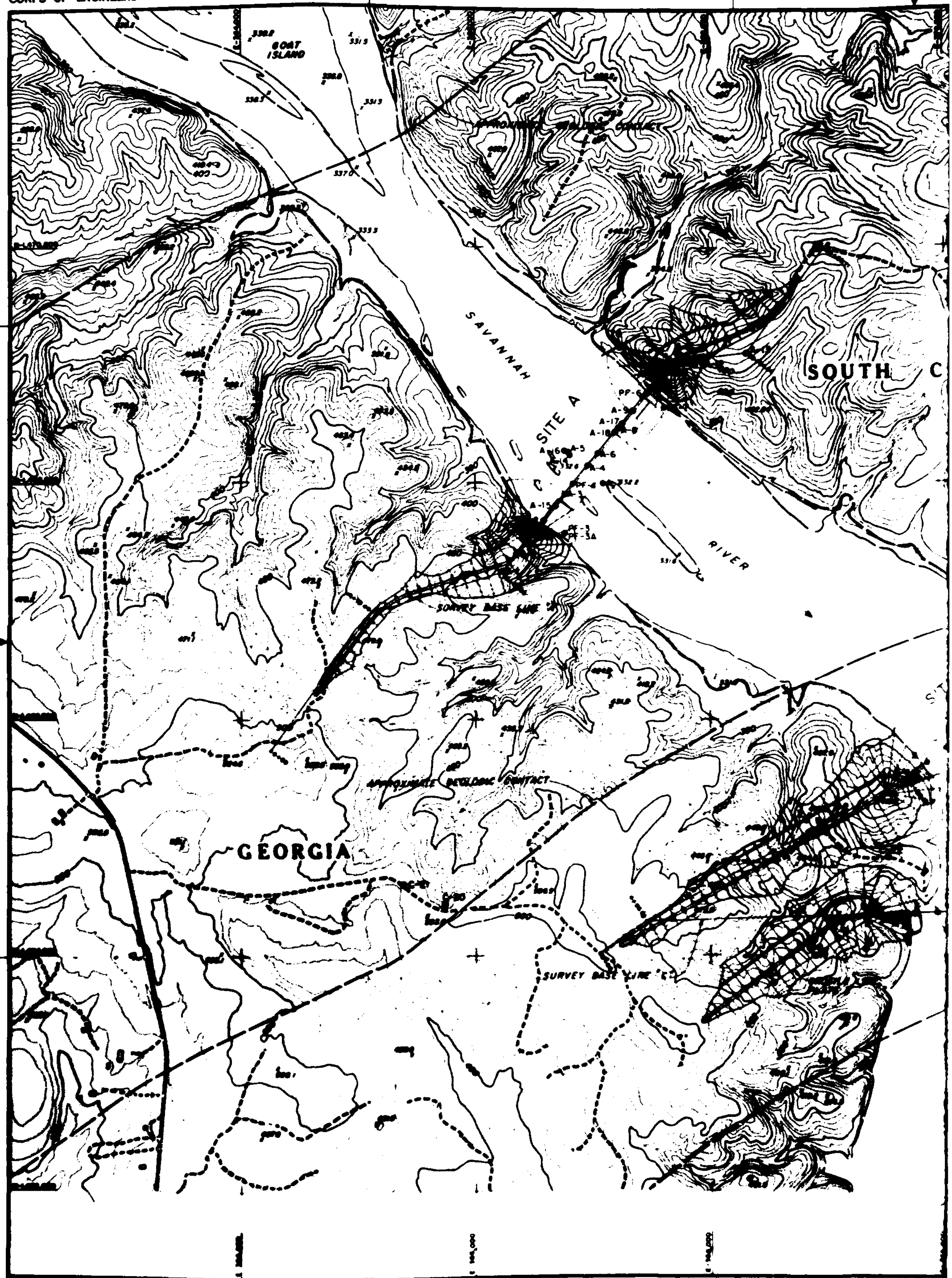
2

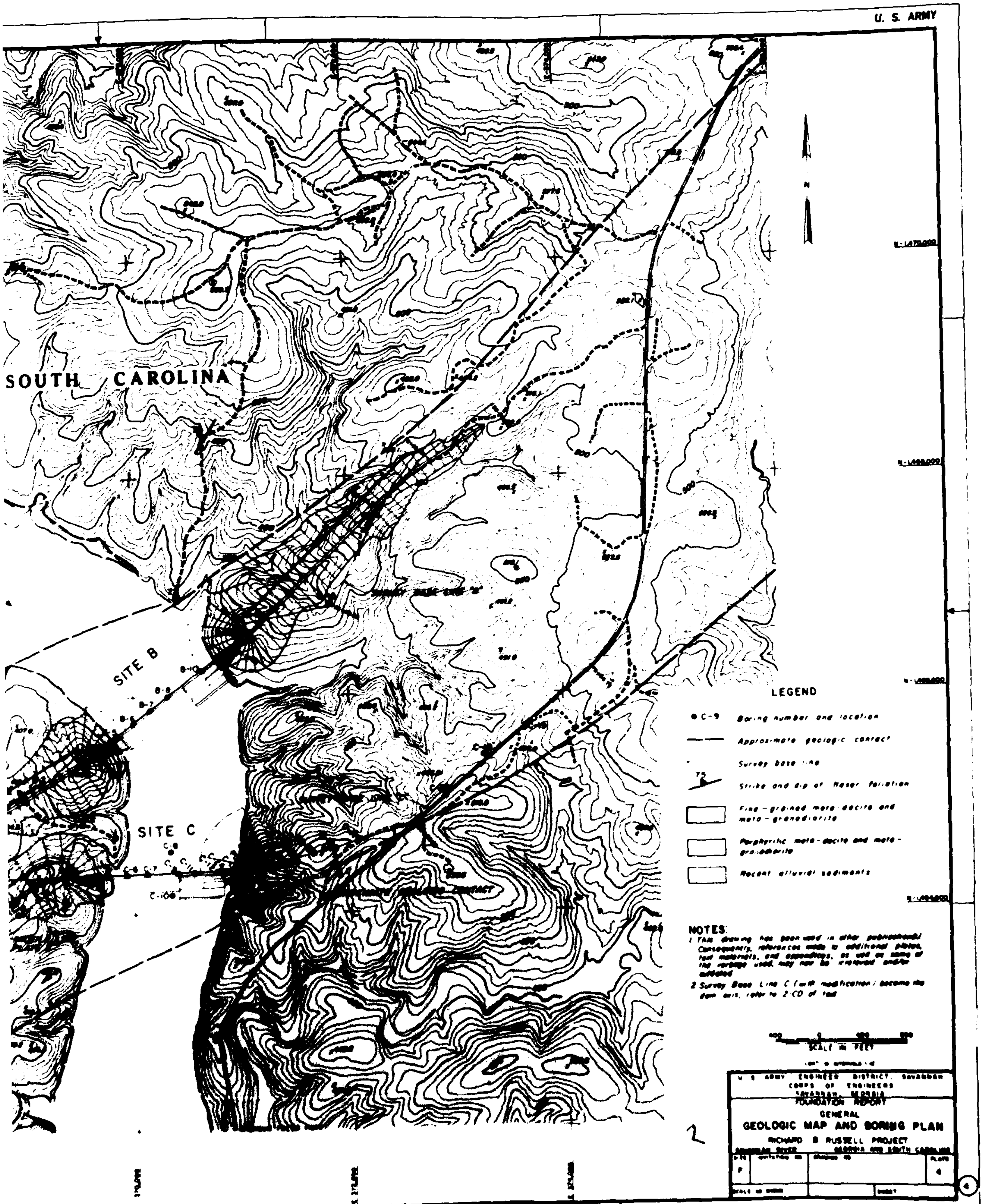
CORPS OF ENGINEERS

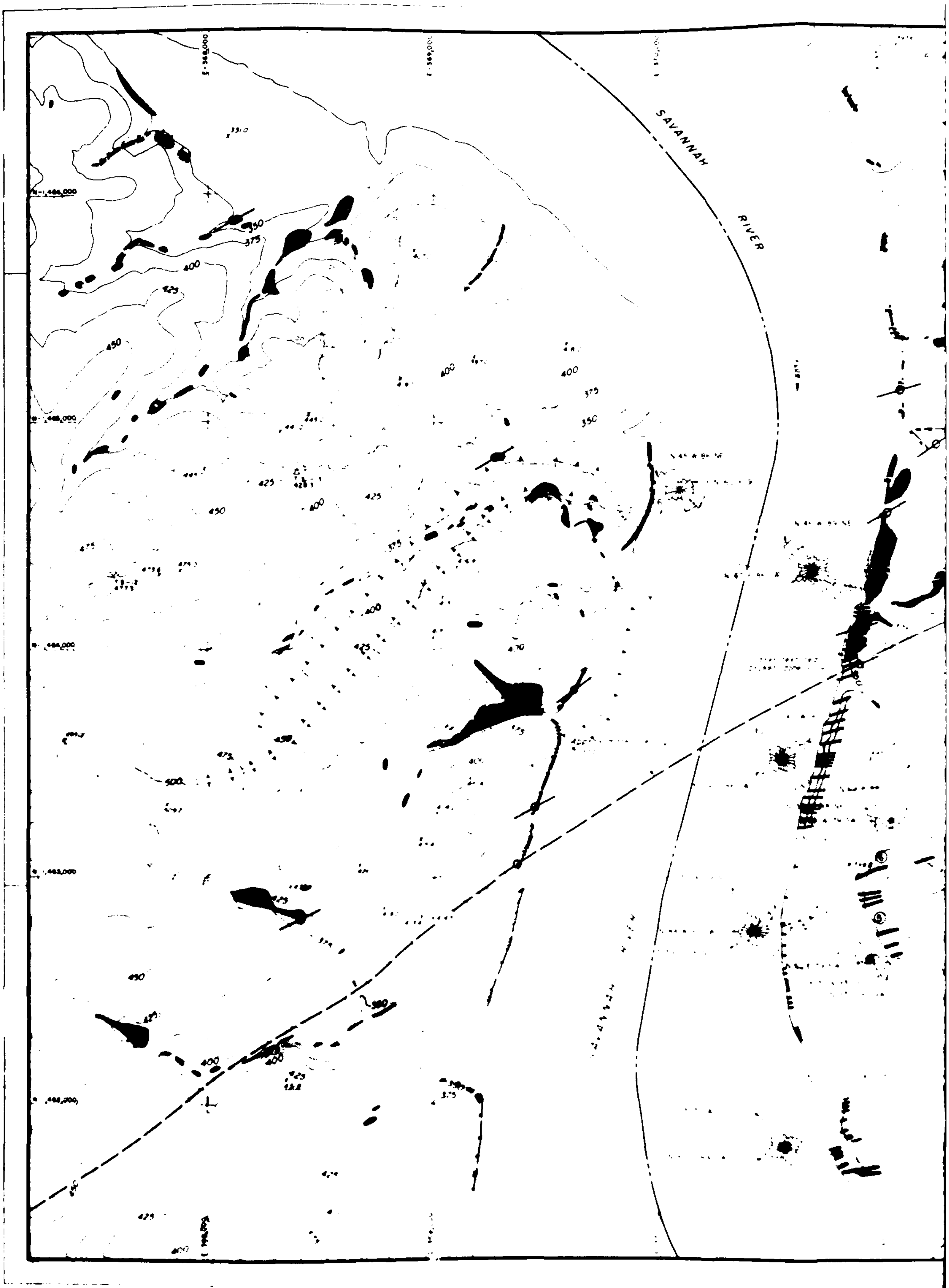



U. S. ARMY
Concrete Dam Aggregate Quarry
Saddle Dike
Fast Access Road
Powerhouse
SAVANNAH RIVER
ABBEVILLE COUNTY
McCORMICK COUNTY
ELBERT COUNTY
CLARK HILL LAKE
SCALE IN FEET
U S ARMY ENGINEER DISTRICT SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
GENERAL
PROJECT SITE PLAN
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
PLATE
3
2

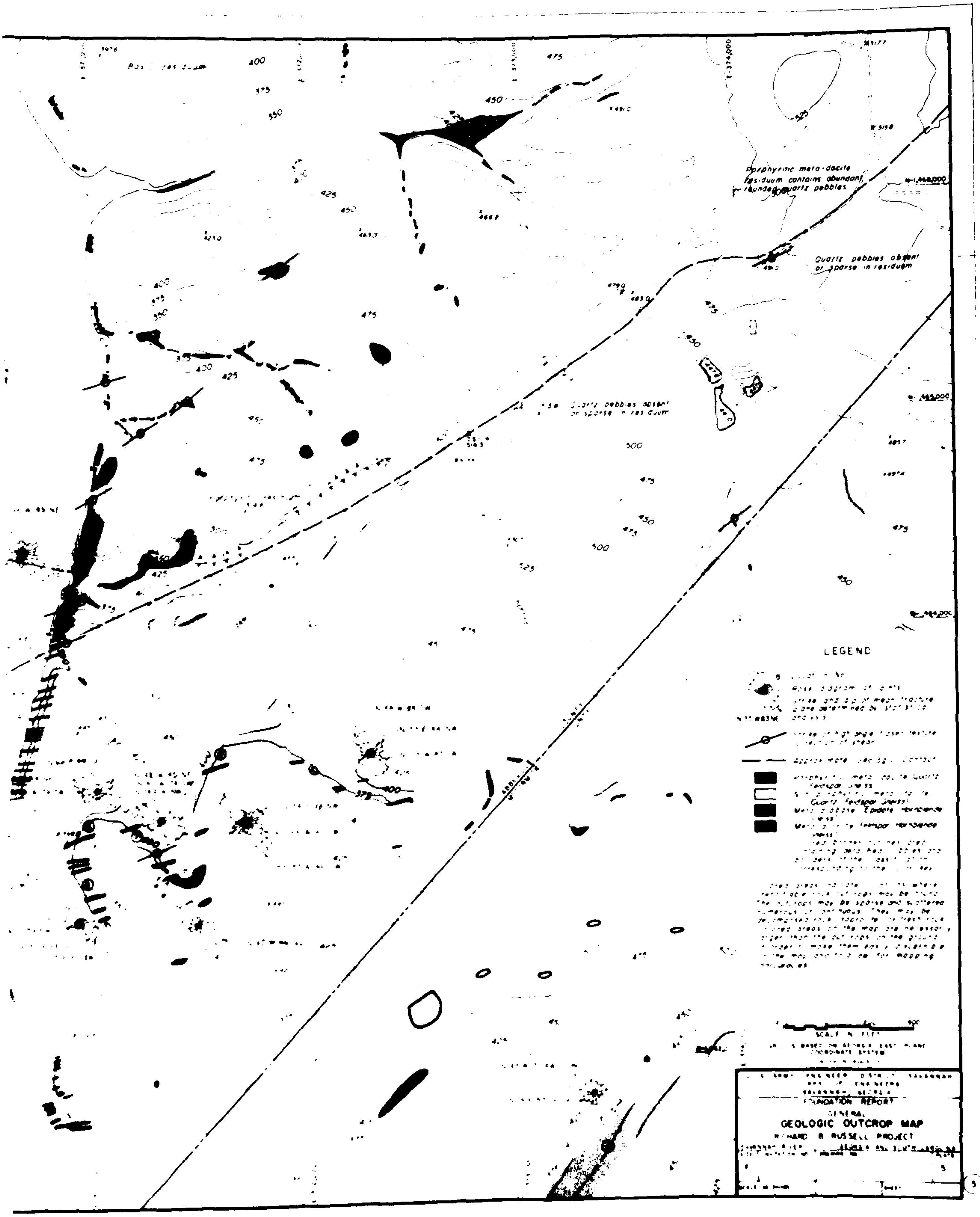
CORPS OF ENGINEERS




U. S. ARMY
SOUTH CAROLINA
SITE B
SITE C
LEGEND
C-9 Boring number and location
Approximate geologic contact
Survey base line
75
Strike and dip of flaser foliation
Fine-grained meta-dacite and meta-granodiorite
Porphyritic meta-dacite and meta-granodiorite
Recent alluvial sediments
NOTES:
2 Survey Base Line C (with modification) become the dam axis, refer to 2-CD of text
SCALE IN FEET
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
GENERAL
GEOLOGIC MAP AND BORING PLAN
RICHARD B RUSSELL PROJECT
PLATE
4
2

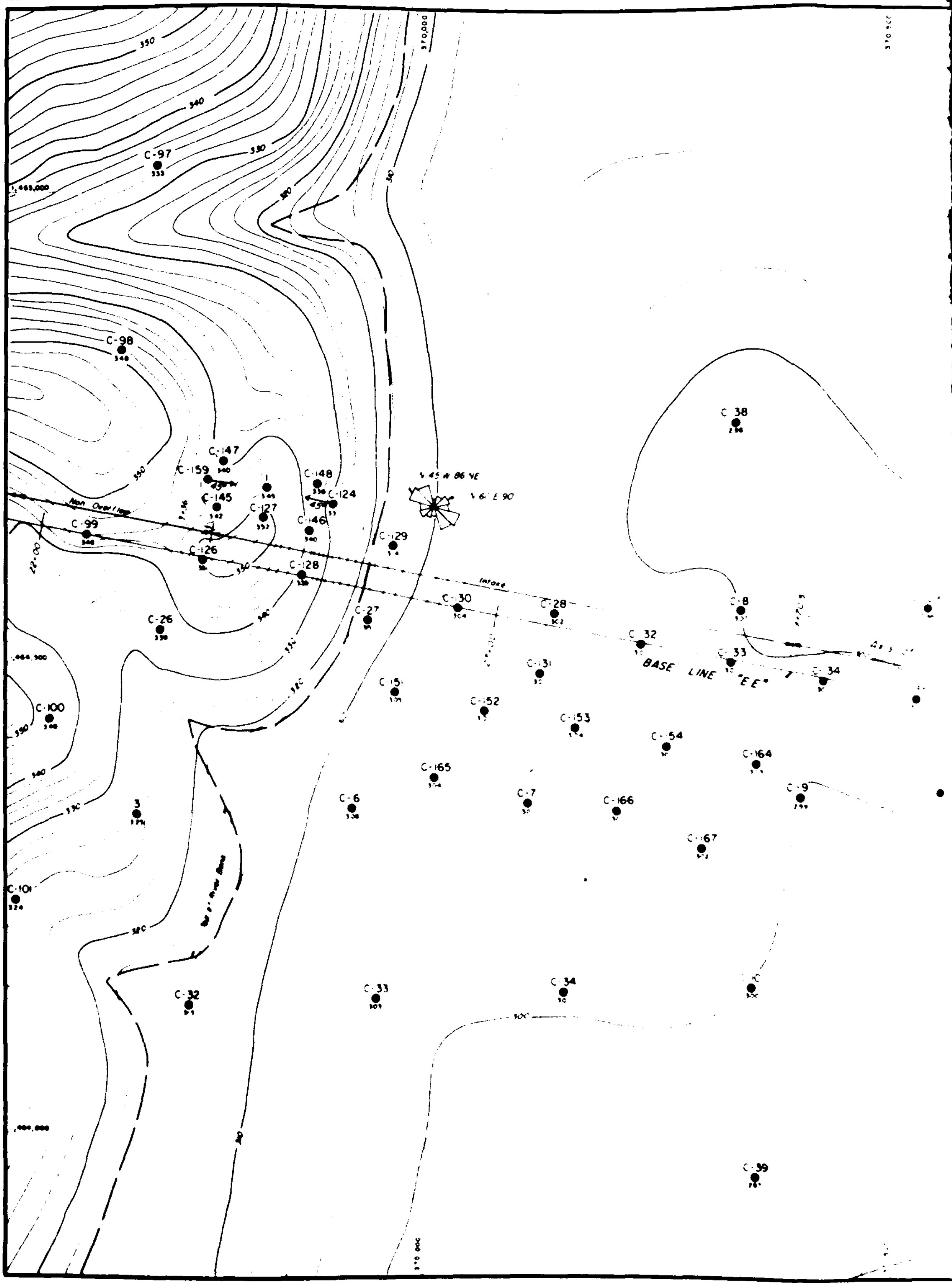


SAVANNAH RIVER
E 568,000
E 569,000
N 1,466,000
400
425
450
475
500
375
350

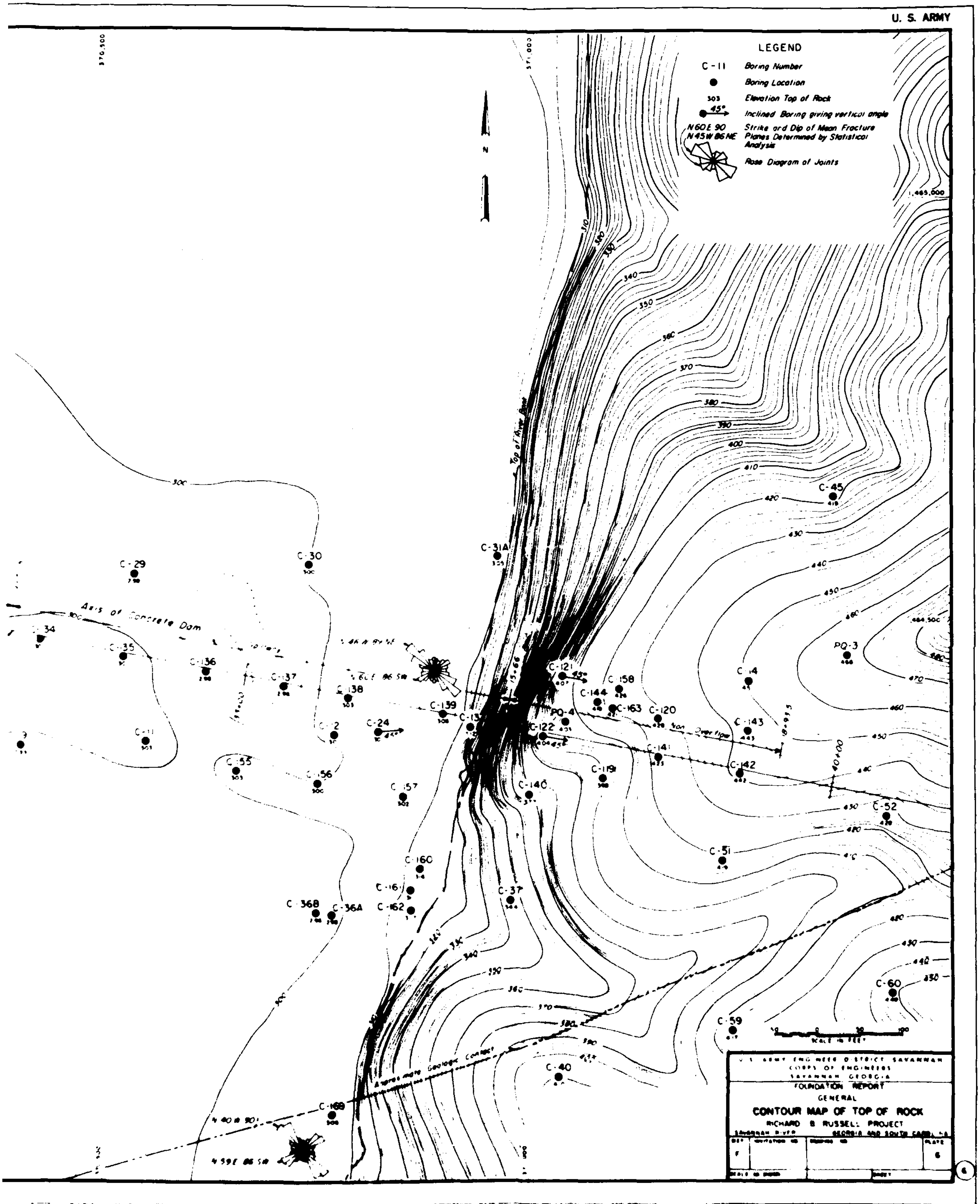


Porphyritic meta-dacite residuum contains abundant rounded quartz pebbles
Quartz pebbles absent or sparse in residuum
Quartz pebbles absent or sparse in residuum
LEGEND
Rose diagram of joints
Approximate geologic contact
Quartz Feldspar Gneiss
Epidote hornblende
SCALE IN FEET
FOUNDATION REPORT
GENERAL
GEOLOGIC OUTCROP MAP
RICHARD B. RUSSELL PROJECT

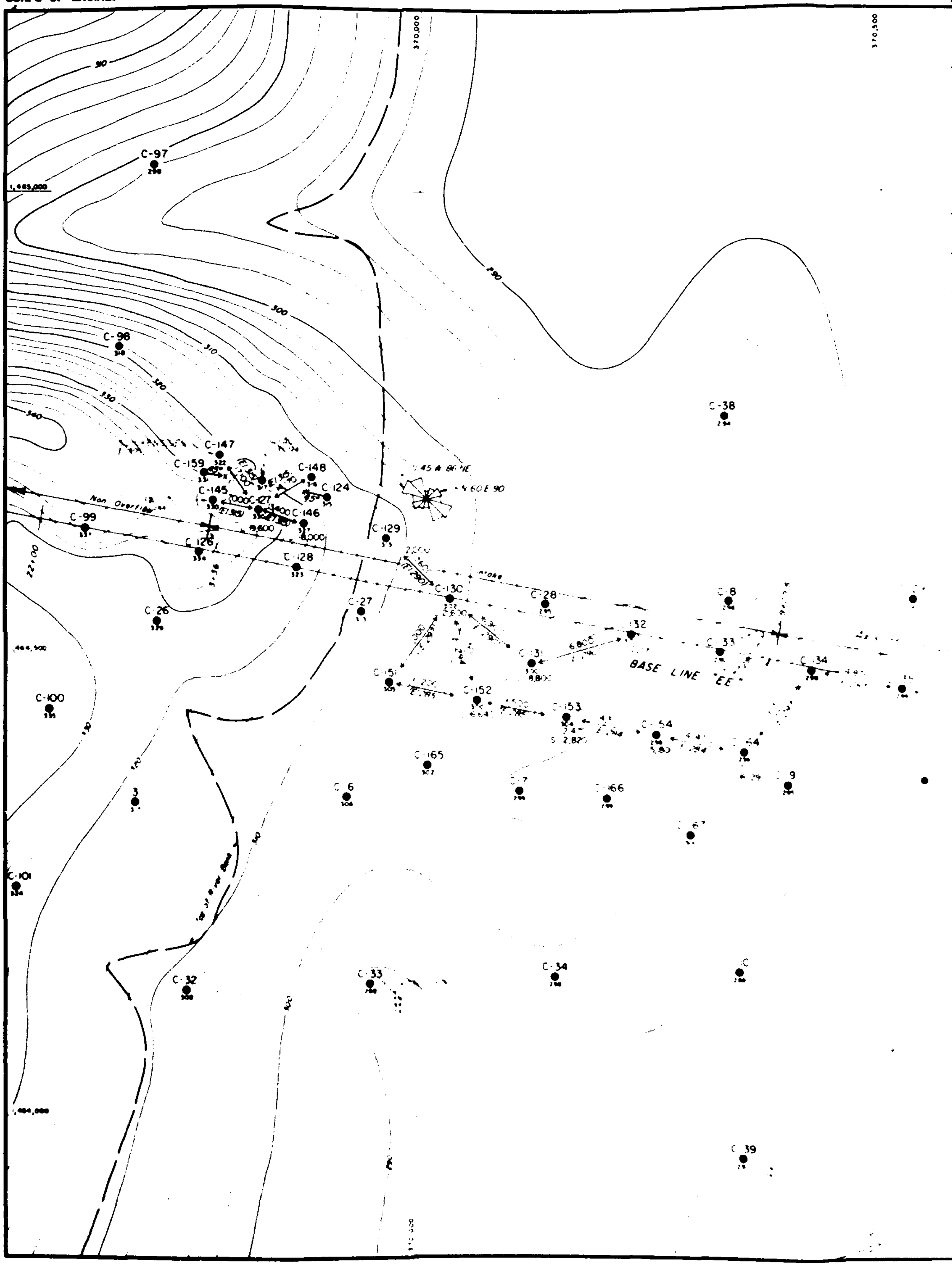
CORPS OF ENGINEERS

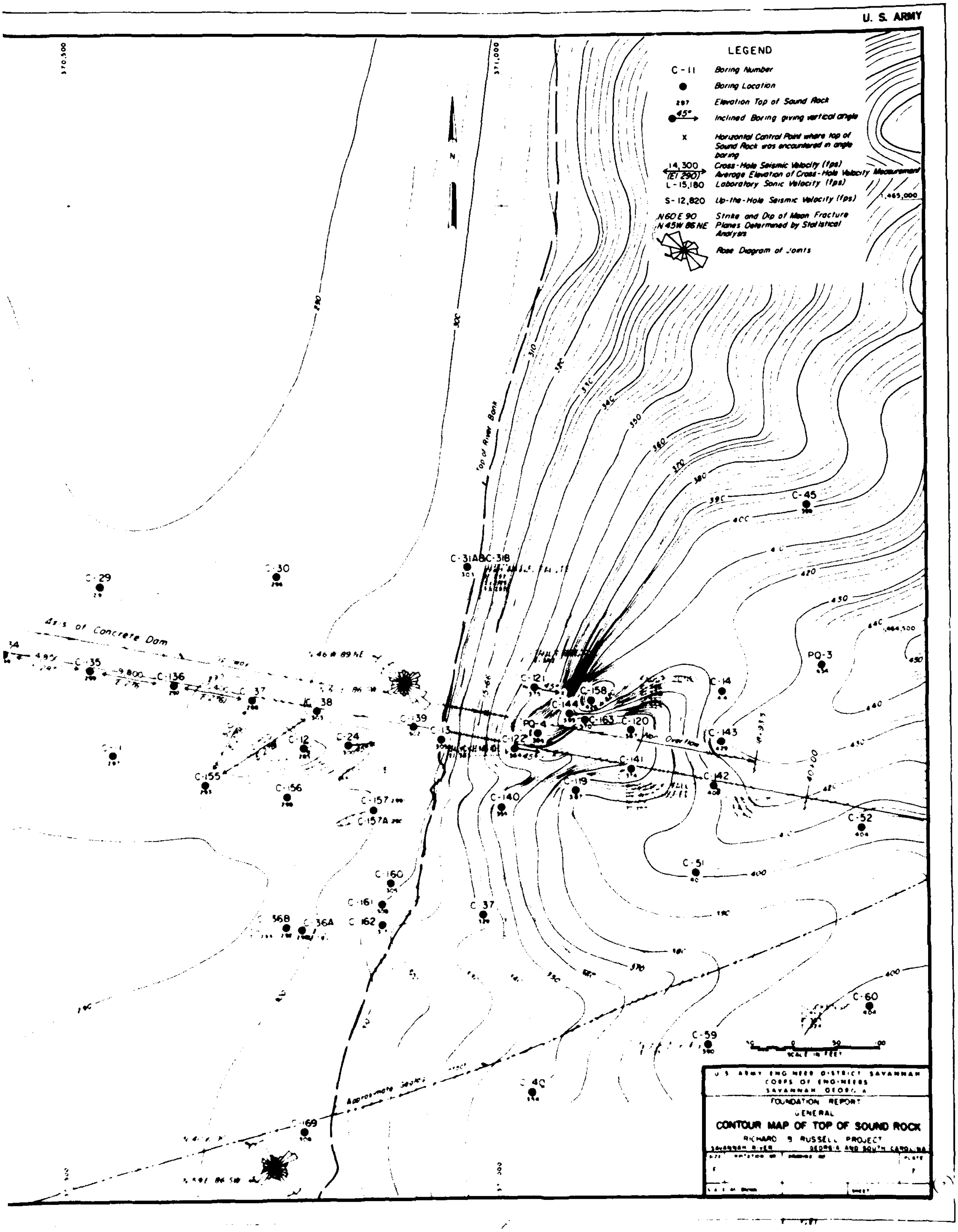


1

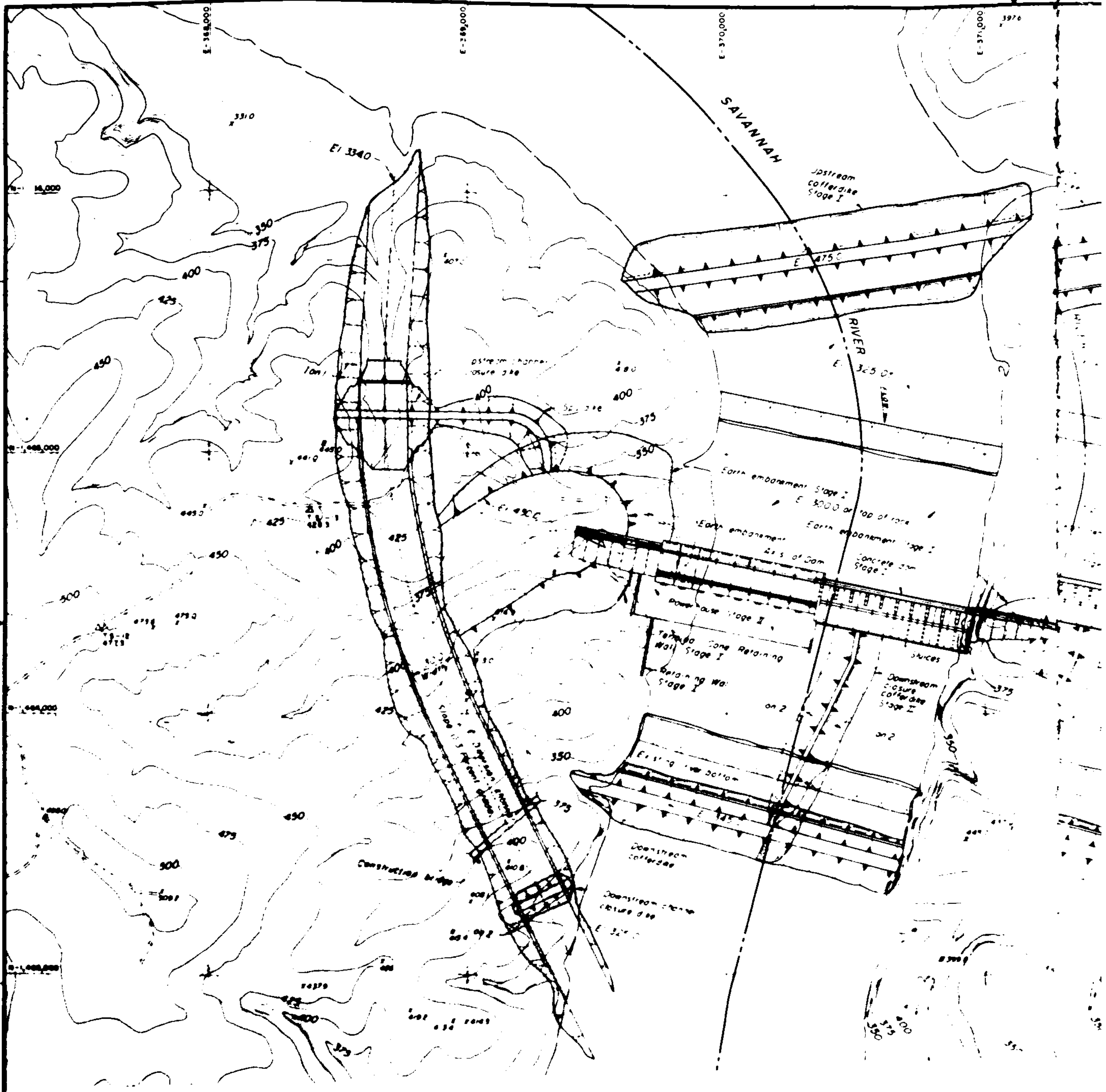

U. S. ARMY
LEGEND
C - 11 Boring Number
Boring Location
303 Elevation Top of Rock
45° Inclined Boring giving vertical angle
N60E 90
N45W 86NE
Strike and Dip of Mean Fracture Planes Determined by Statistical Analysis
Rose Diagram of Joints
N
Axis of Concrete Dam
Top of River Bank
Non Overflow
Approximate Geologic Contact
C-29
C-30
C-31A
C-34
C-35
C-136
C-137
C-138
C-139
C-11
C-2
C-24
C-55
C-56
C-57
C-121
C-158
C-144
C-163
C-120
C-14
C-143
PQ-4
C-122
C-141
C-142
C-119
C-140
C-45
PQ-3
C-52
C-51
C-160
C-161
C-162
C-36B
C-36A
C-37
C-59
C-60
C-40
C-169
300
310
320
330
340
350
360
370
380
390
400
410
420
430
440
450
460
470
SCALE IN FEET
U.S. ARMY ENGINEER DISTRICT SAVANNAH
CORPS OF ENGINEERS
SAVANNAH GEORGIA
FOUNDATION REPORT
GENERAL
CONTOUR MAP OF TOP OF ROCK
RICHARD B RUSSELL PROJECT
PLATE
6

CORPS OF ENGINEERS



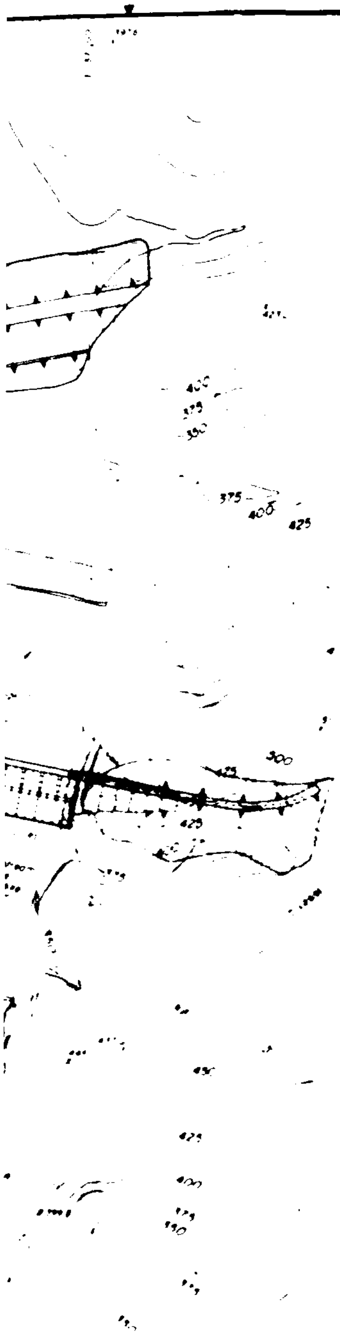

U. S. ARMY
LEGEND
C - 11 Boring Number
Boring Location
297 Elevation Top of Sound Rock
45° Inclined Boring giving vertical angle
x Horizontal Control Point where top of Sound Rock was encountered in angle boring
14,300 Cross-Hole Seismic Velocity (fps)
(El 290) Average Elevation of Cross-Hole Velocity Measurement
L-15,180 Laboratory Sonic Velocity (fps)
S-12,820 Up-the-Hole Seismic Velocity (fps)
N60E 90
N45W 86NE Strike and Dip of Mean Fracture Planes Determined by Statistical Analysis
Rose Diagram of Joints
370,500
371,000
1,465,000
1,464,500
N
Top of River Bank
Axis of Concrete Dam
Non Overflow
Approximate Sewer
C-29
C-30
C-31A
C-31B
C-35
C-136
C-37
C-38
C-139
C-13
C-12
C-24
C-155
C-156
C-157
C-157A
C-121
C-158
C-144
C-163
C-120
PQ-4
C-122
C-141
C-119
C-140
C-14
C-143
C-142
C-45
PQ-3
C-52
C-51
C-160
C-161
C-162
C-36B
C-36A
C-37
C-60
C-59
C-40
C-169
290
300
310
350
360
370
380
420
430
440
400
SCALE IN FEET
U S ARMY ENGINEER DISTRICT SAVANNAH
CORPS OF ENGINEERS
SAVANNAH GEORGIA
FOUNDATION REPORT
GENERAL
CONTOUR MAP OF TOP OF SOUND ROCK
RICHARD B RUSSELL PROJECT

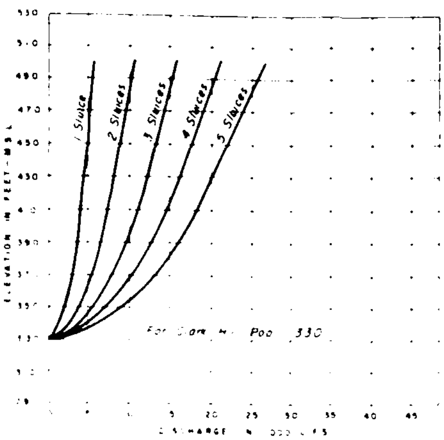
CORPS OF ENGINEERS



PLAN

SCALE





RATING CURVES - OUTLET SLUICES

NOTES

1. The diversion channel was excavated and upstream and downstream cofferdikes constructed across the river for Stage I construction.
2. The concrete dam, powerhouse, retaining wall and part of the West Georgia terminal cone will be constructed [illegible] Stage I. The terminal cone with some earth embankment will be constructed to [illegible]
3. The downstream closure cofferdike will be constructed for Stage II Diversion.
4. The upstream cofferdike will be breached on the east side and east part of the downstream cofferdike removed. The river will then be diverted through the sluices of the concrete dam or over the spillway for Stage II construction.
5. The diversion channel will have upstream and downstream closure dikes placed prior to completion of the Georgia earth embankment.

NOTE

[illegible]

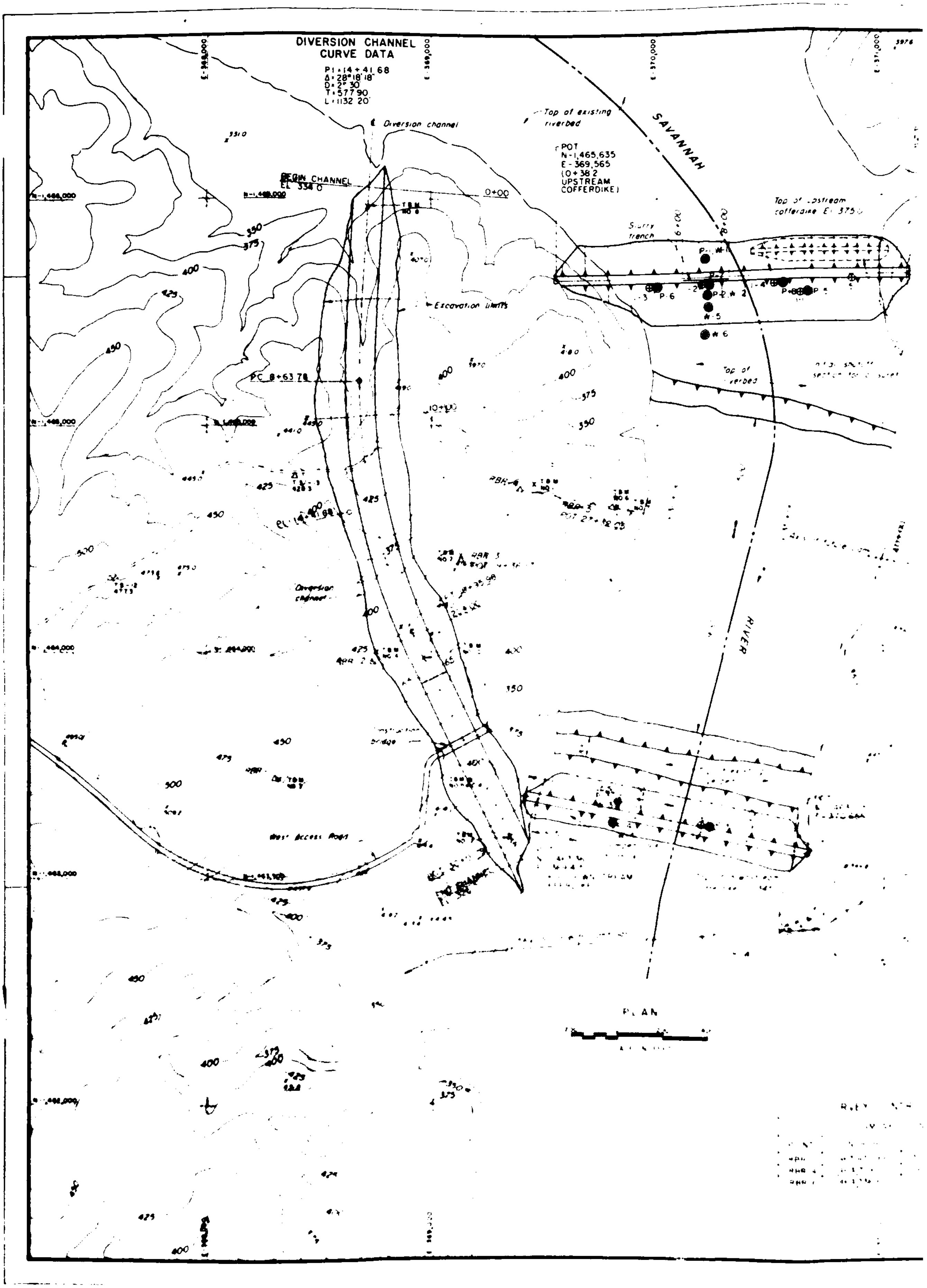
U S ARMY

REVISIONS

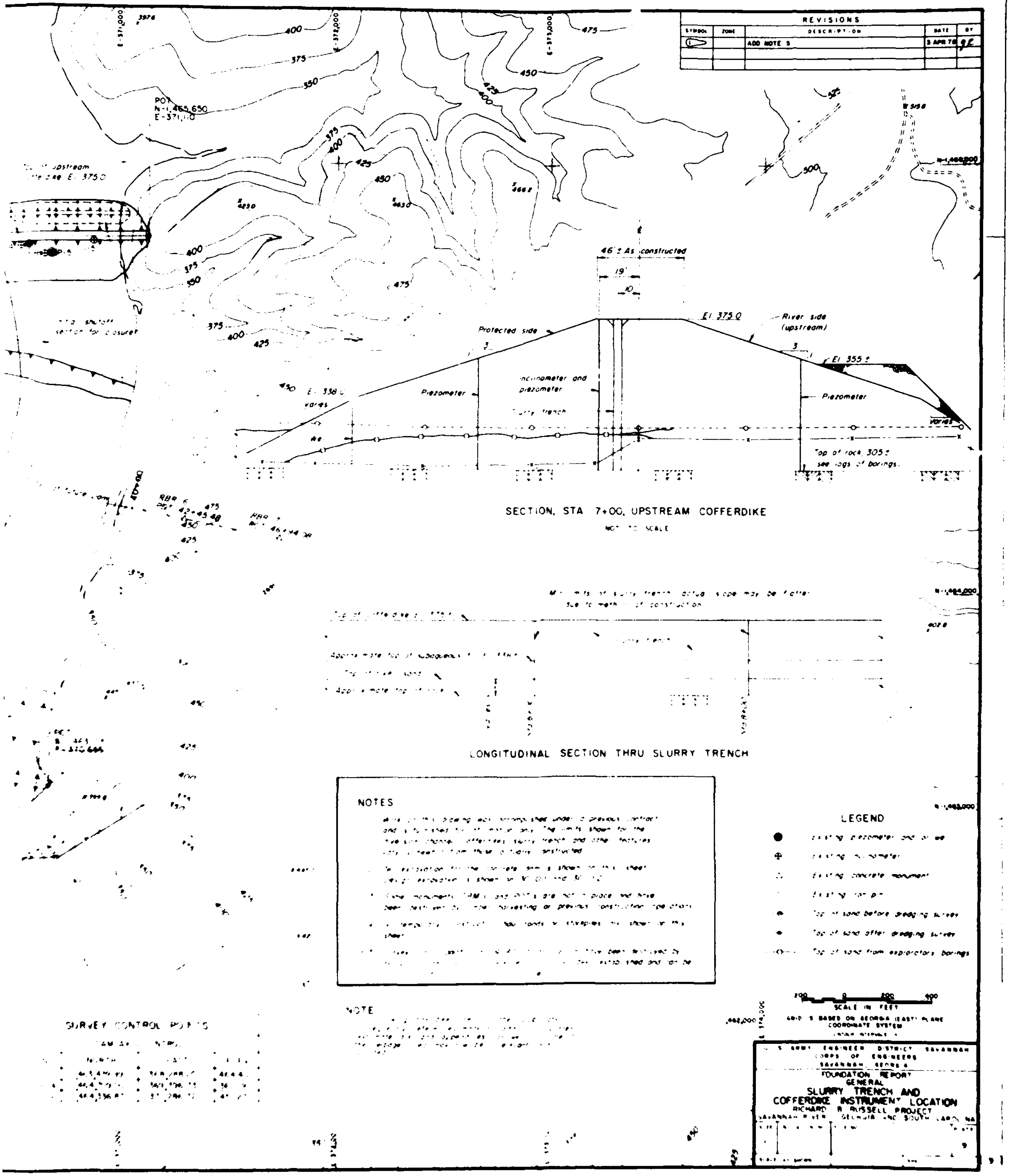
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

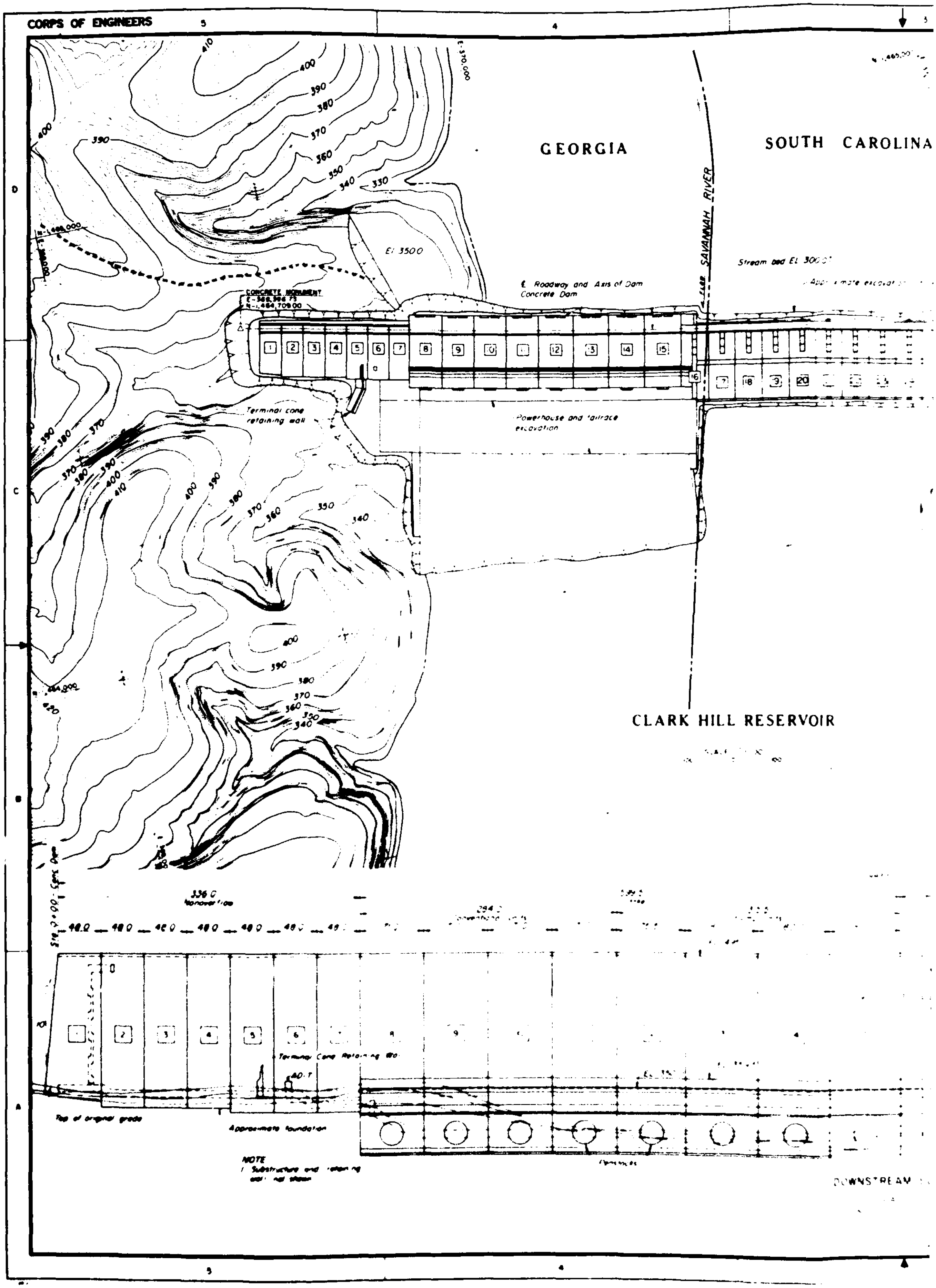
FOUNDATION REPORT
GENERAL
GENERAL RIVER DIVERSION PLAN
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

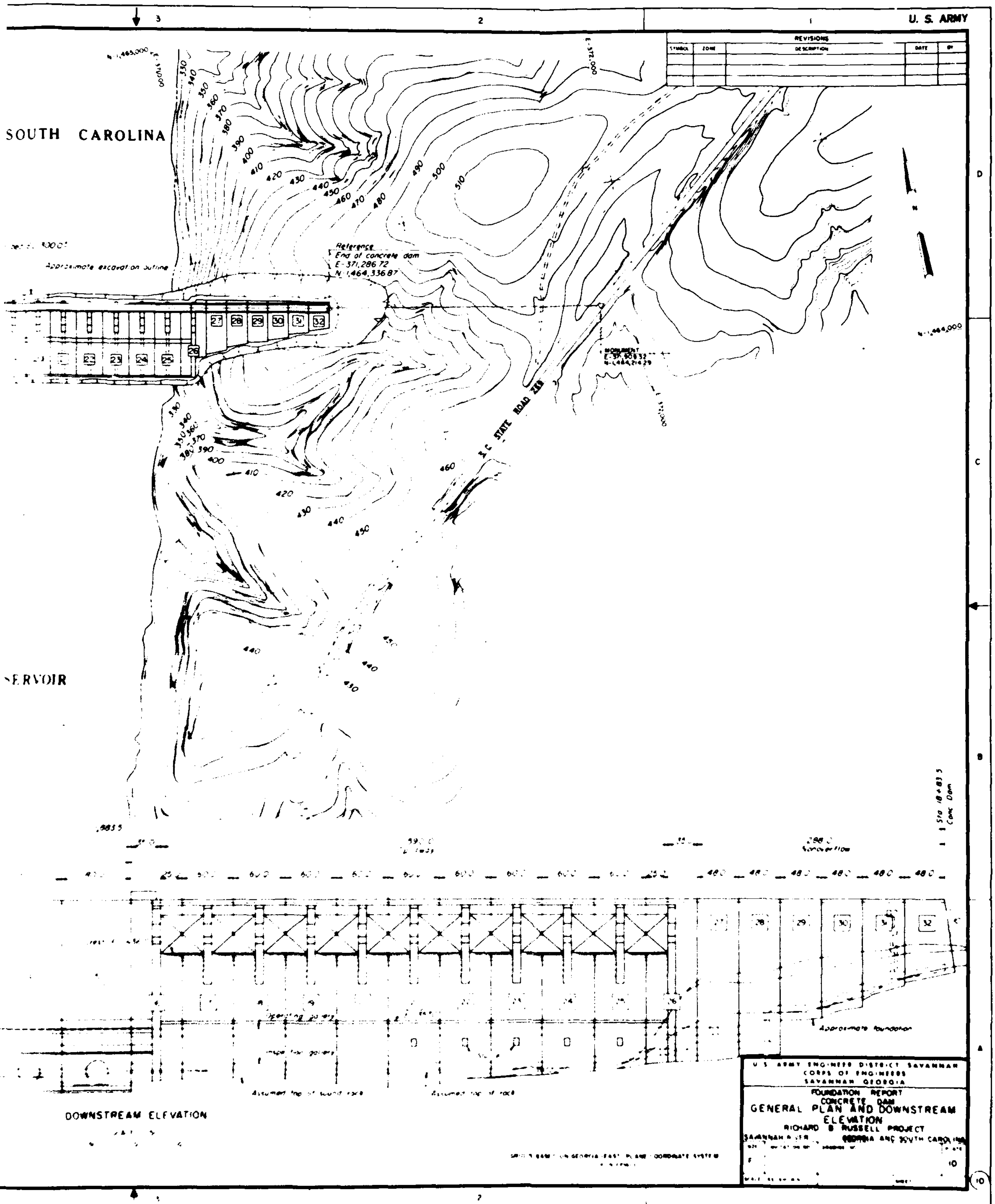
8



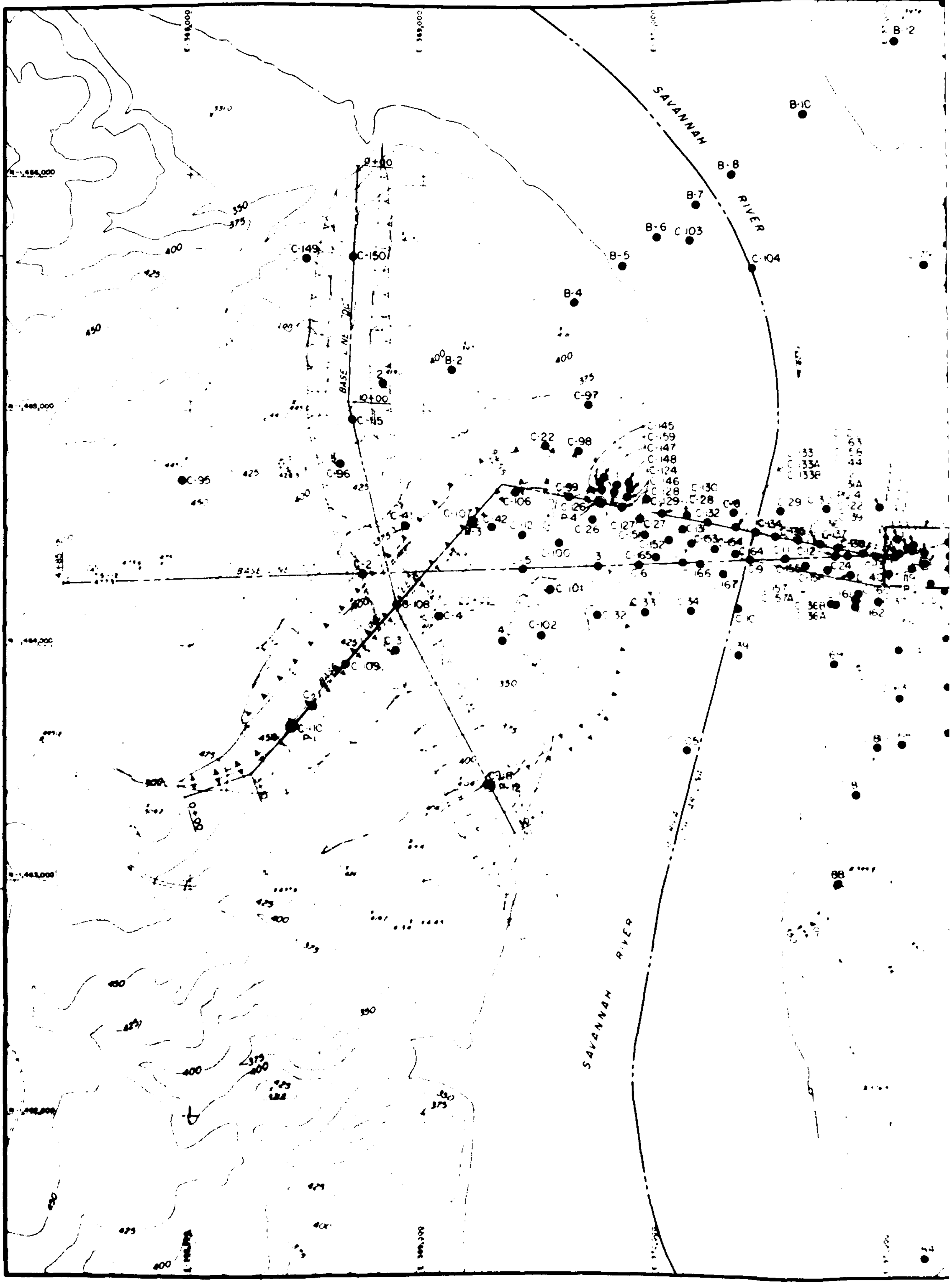
DIVERSION CHANNEL
CURVE DATA
PI = 14 + 41.68
Δ = 28°18'18"
D = 2°30'
T = 577.90
L = 1132.20'
Diversion channel
Top of existing riverbed
SAVANNAH
RIVER
POT
N-1,465,635
E-369,565
(0+38.2
UPSTREAM
COFFERDIKE)
BEGIN CHANNEL
EL 334.0
0+00
Slurry trench
Top of upstream cofferdike El 375.0
Excavation limits
PC 8+63.78
10+00
Top of riverbed
Diversion channel
Construction bridge
West Access Road
PLAN

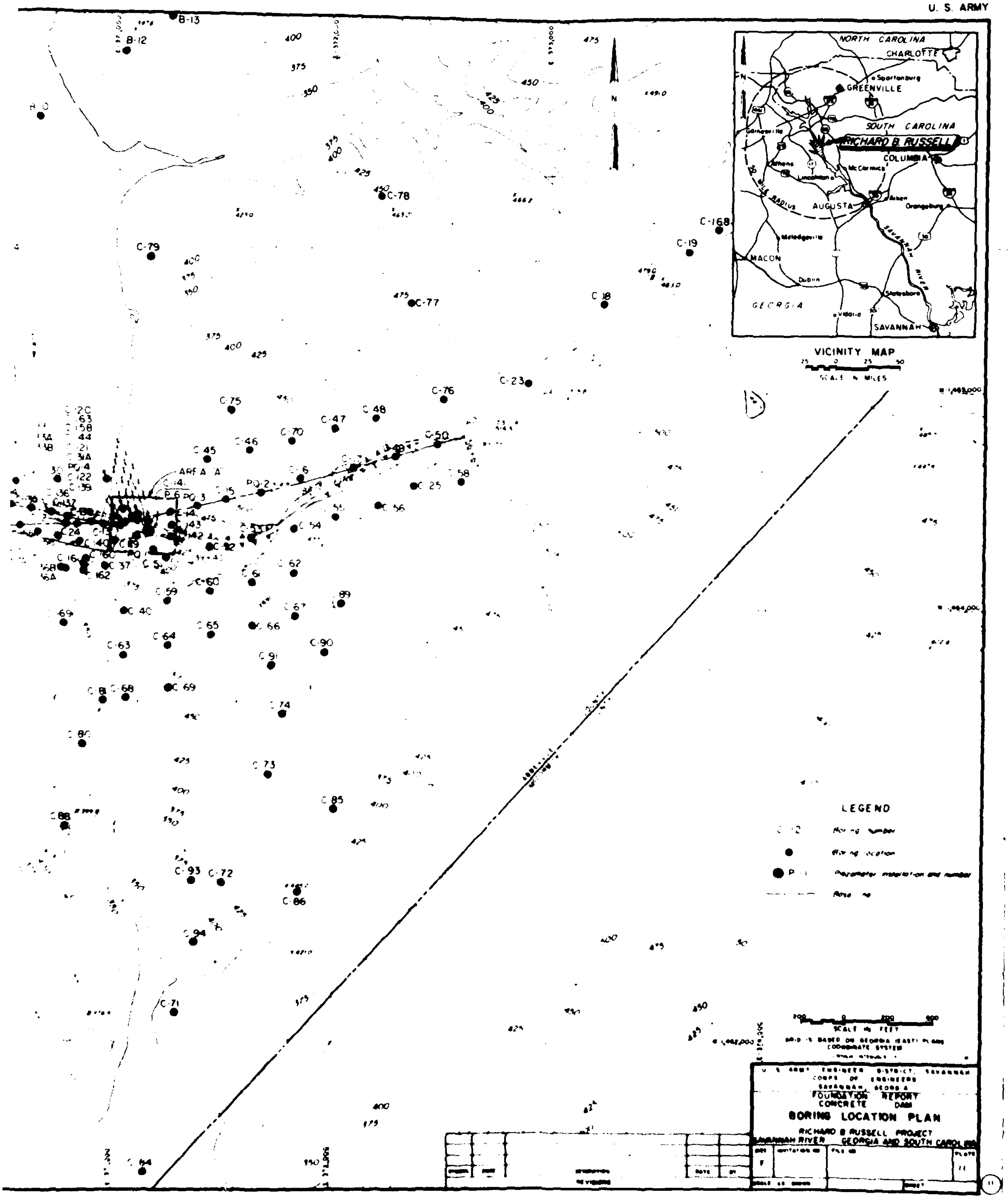

REVISIONS
ADD NOTE 5
SECTION, STA 7+00, UPSTREAM COFFERDIKE
NOT TO SCALE
46'± As constructed
19'
10'
El. 375.0
River side (upstream)
Protected side
El. 355±
Inclinometer and piezometer
Piezometer
El. 338.0 varies
Top of rock 305± see logs of borings.
LONGITUDINAL SECTION THRU SLURRY TRENCH
NOTES
LEGEND
Existing concrete monument
Top of sand before dredging survey
Top of sand after dredging survey
Top of sand from exploratory borings
SCALE IN FEET
SURVEY CONTROL POINTS
NOTE
U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
GENERAL
SLURRY TRENCH AND
COFFERDIKE INSTRUMENT LOCATION
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER, GEORGIA AND SOUTH CAROLINA

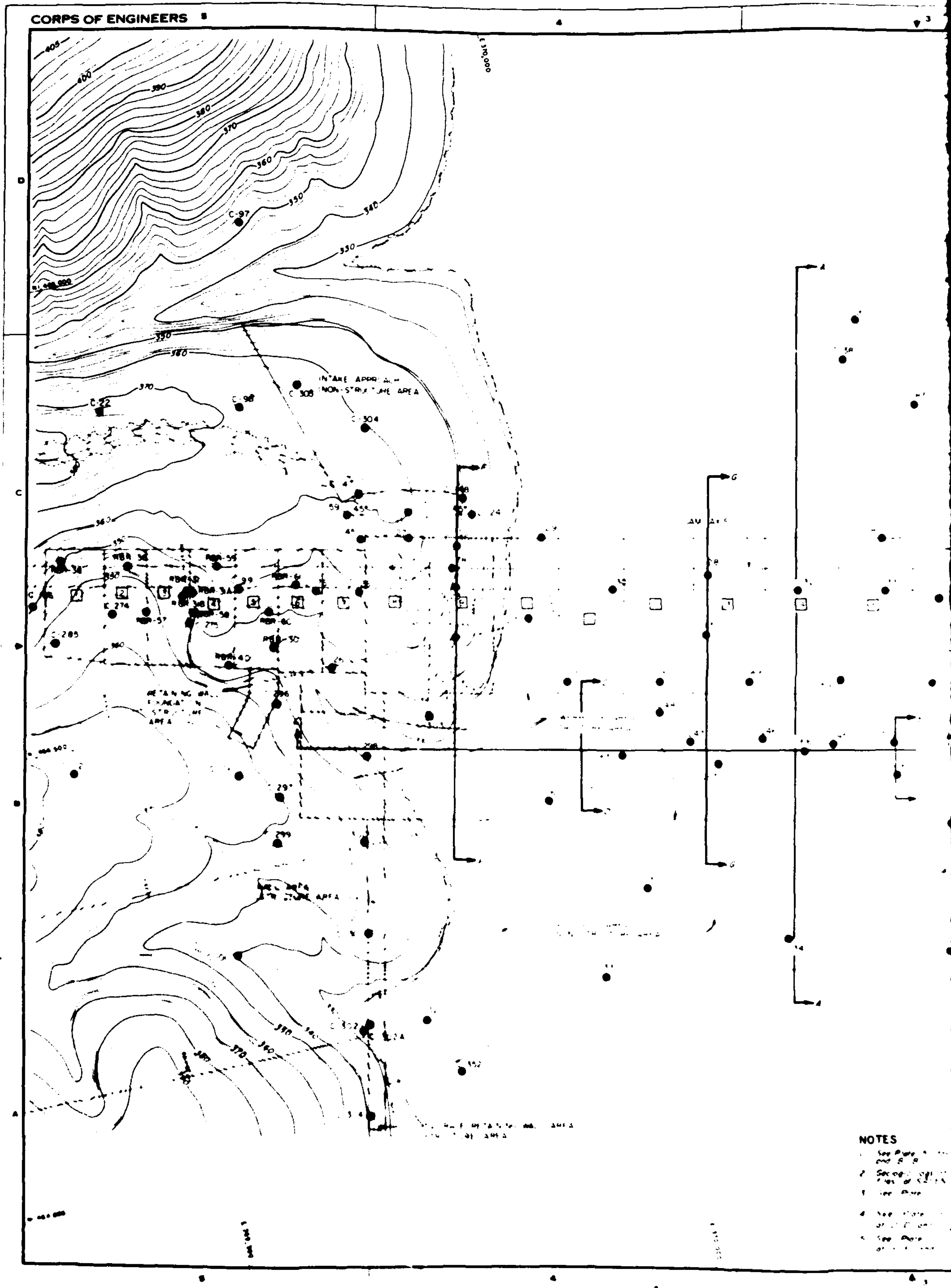

CORPS OF ENGINEERS
GEORGIA
SOUTH CAROLINA
SAVANNAH RIVER
El 350.0
Stream bed El 300.0
℄ Roadway and Axis of Dam Concrete Dam
CONCRETE MONUMENT
Terminal cone retaining wall
Powerhouse and tailrace excavation
CLARK HILL RESERVOIR
336.0
Nonoverflow
Terminal Cone Retaining Wall
Top of original grade
Approximate foundation
NOTE
1 Substructure and retaining wall not shown
DOWNSTREAM

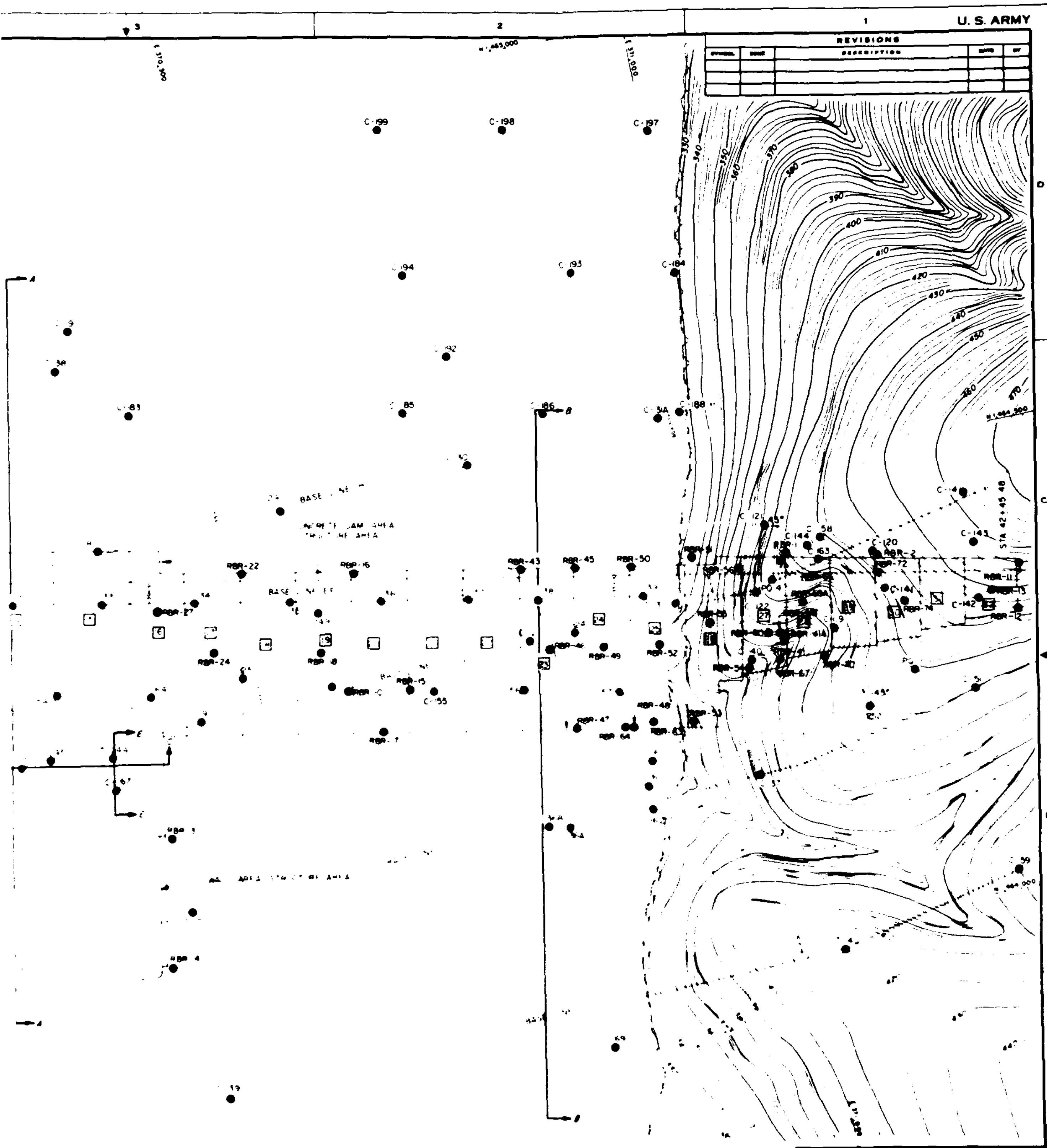

U. S. ARMY
REVISIONS
SYMBOL
ZONE
DESCRIPTION
DATE
BY
SOUTH CAROLINA
E-372,000
Reference
End of concrete dam
E-371,286.72
N-1,464,336.87
Approximate excavation outline
MONUMENT
S. C. STATE ROAD 284
N-1,464,000
SERVOIR
Approximate foundation
Operating gallery
Inspection gallery
Assumed top of sound rock
Assumed top of rock
DOWNSTREAM ELEVATION
U. S. ARMY ENGINEER DISTRICT SAVANNAH
CORPS OF ENGINEERS
SAVANNAH GEORGIA
FOUNDATION REPORT
CONCRETE DAM
GENERAL PLAN AND DOWNSTREAM
ELEVATION
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
10

CORPS OF ENGINEERS




U. S. ARMY
NORTH CAROLINA
CHARLOTTE
Spartanburg
GREENVILLE
SOUTH CAROLINA
RICHARD B. RUSSELL
Gainesville
Athens
COLUMBIA
McCormick
Lincolnton
AUGUSTA
Aiken
Orangeburg
Milledgeville
MACON
Dublin
GEORGIA
Vidalia
Statesboro
SAVANNAH
VICINITY MAP
SCALE IN MILES
LEGEND
Boring number
Boring location
Piezometer installation and number
B-13
B-12
C-78
C-79
C-77
C-168
C-19
C-18
C-23
C-75
C-76
C-47
C-48
C-70
C-46
C-45
C-50
AREA "A"
PO-2
PO 3
C 25
C 58
C 56
C 54
C 62
C 60
C 59
C 40
C 67
C 66
C 65
C 64
C 63
C 90
C 69
C 68
C 74
C 73
C 85
C 88
C-93
C-72
C-86
C-94
C-71
SCALE IN FEET
U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
CONCRETE DAM
BORING LOCATION PLAN
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
REVISIONS


CORPS OF ENGINEERS
405
400
390
380
370
360
350
340
C-97
C-98
C-22
C-308
C-304
INTAKE APPROACH
NON-STRUCTURE AREA
RBR-38
RBR-55
RBR-57
RBR-6C
RBR-40
C-285
C-352
NOTES



NOTES

1. See Plate 3 for geologic cross sections of A A and B B
2. Geologic logs of borings are maintained in the files of SASEN-GG
3. See Plate 11 for geologic profile of C C
4. See Plate 14 for geologic cross sections of D D and E E
5. See Plate for geologic cross sections of F F and G G

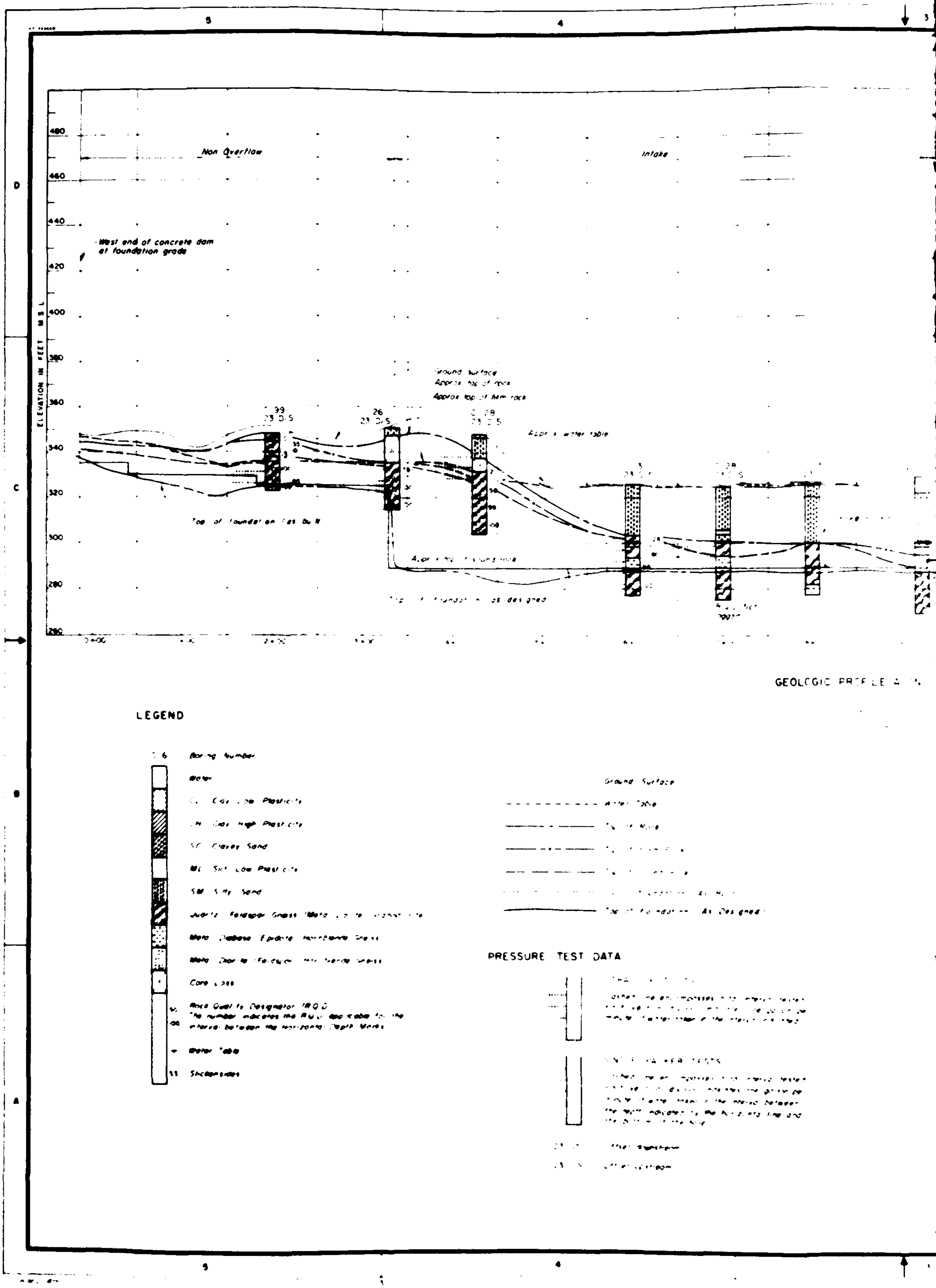
SCALE IN FEET

U S ARMY ENGINEER DISTRICT SAVANNAH
CORPS OF ENGINEERS

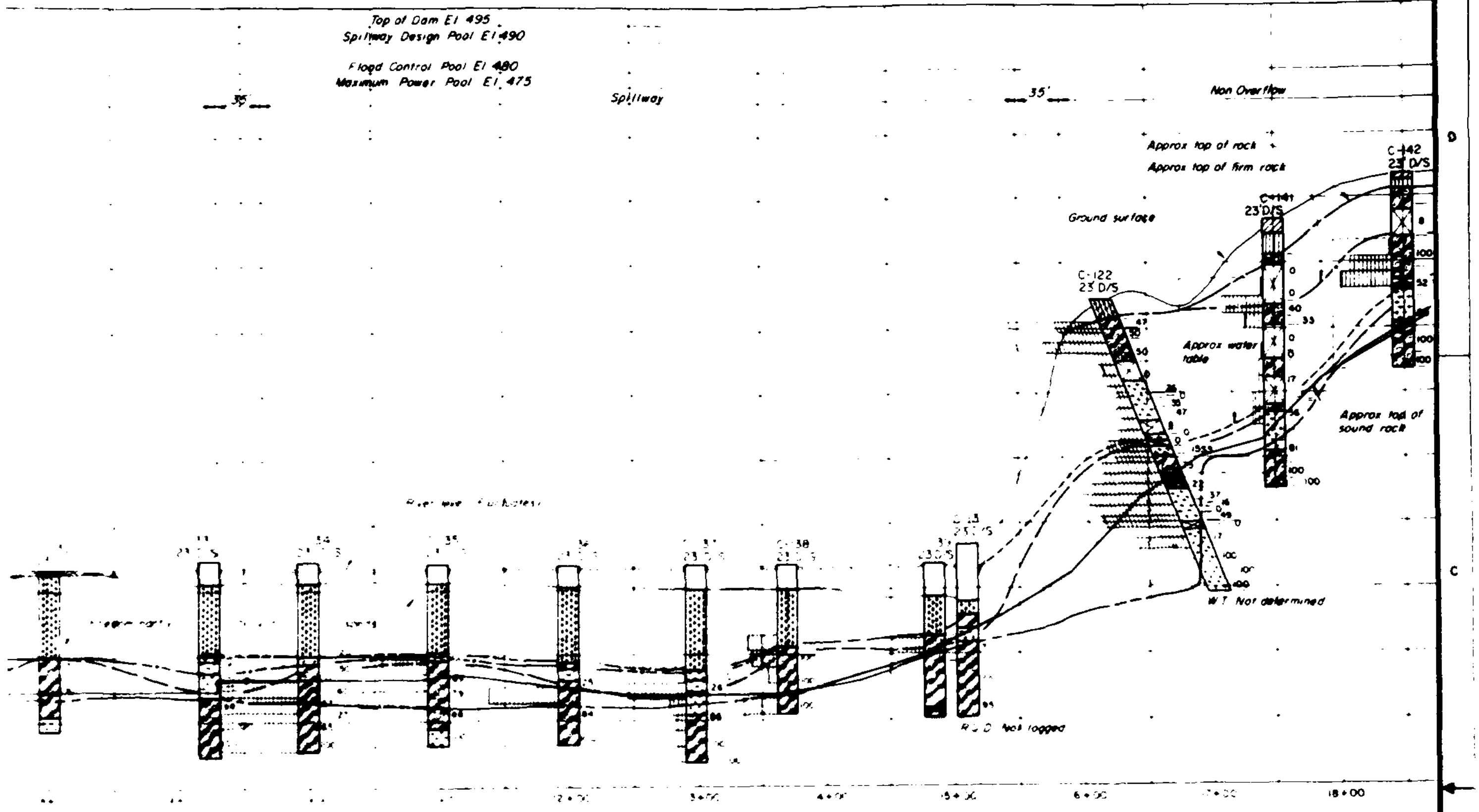
FOUNDATION REPORT
CONCRETE DAM
PLAN OF BORINGS
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

12

2



Non Overflow
Intake
West end of concrete dam at foundation grade
ELEVATION IN FEET M.S.L.
Ground surface
Approx top of rock
Approx top of firm rock
Approx. water table
Top of foundation (as built)
Top of foundation as designed
GEOLOGIC PROFILE
LEGEND
Boring Number
Water
Core Loss
Rock Quality Designator (R.Q.D.)
The number indicates the R.Q.D. applicable for the interval between the horizontal Depth Marks
Water Table
Slickensides
Ground Surface
PRESSURE TEST DATA



GEOLOGIC PROFILE ALONG AXIS OF CONCRETE DAM

NOTES

1. See Plate 2 for location of borings
2. Geologic logs of borings are maintained in the files of SASEN GG

U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM

GEOLOGIC PROFILE WITH AS-BUILT/ AS-DESIGNED FOUNDATION GRADE LINES

RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

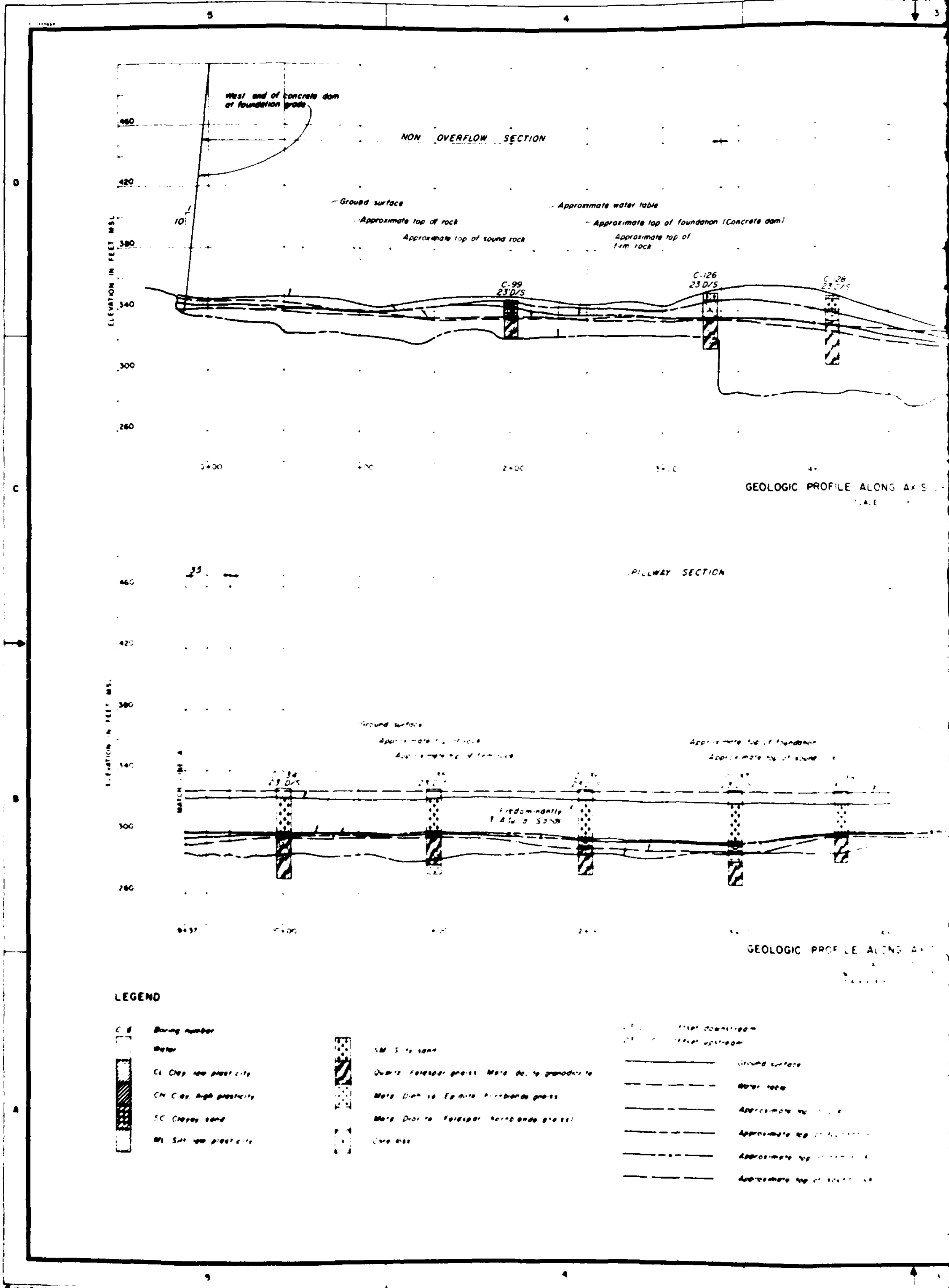
PLATE 13

REVISIONS

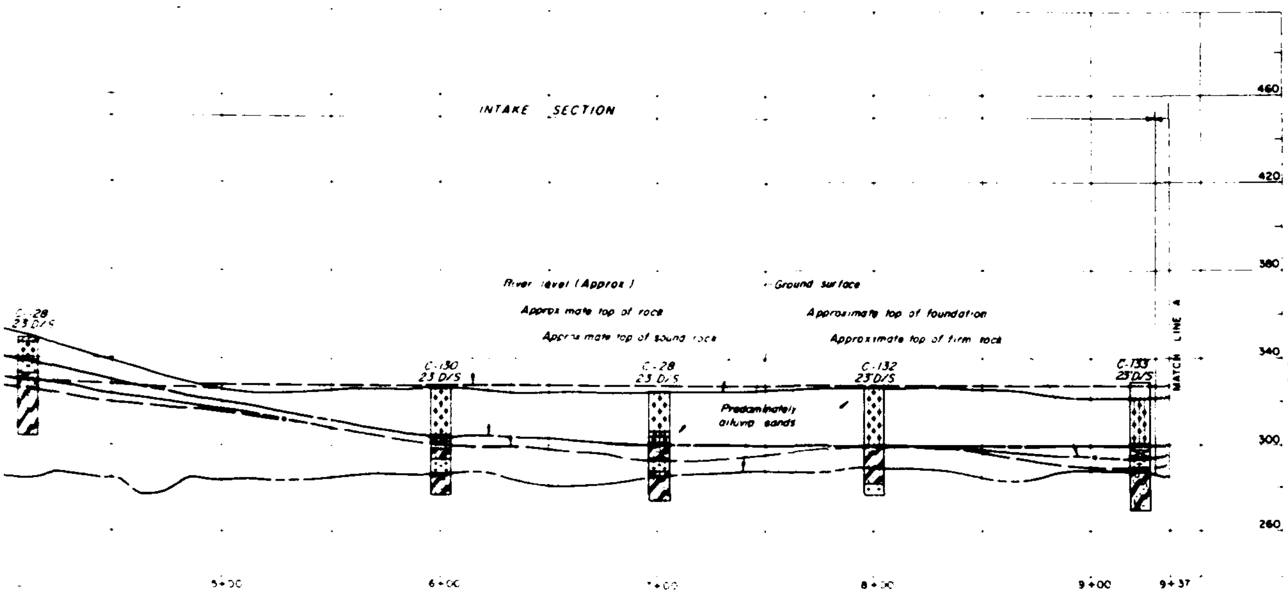
13

2

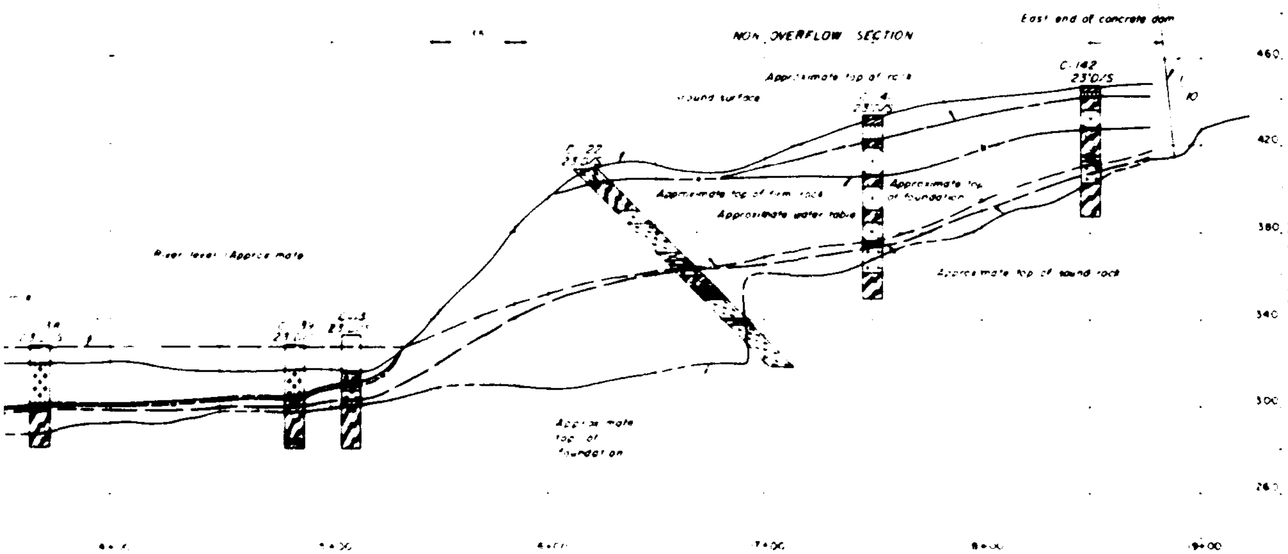
2 ok



West end of concrete dam at foundation grade
NON OVERFLOW SECTION
Ground surface
Approximate top of rock
Approximate top of sound rock
Approximate water table
Approximate top of foundation (Concrete dam)
Approximate top of firm rock
ELEVATION IN FEET MSL
460
420
380
340
300
260
C-99 23'D/S
C-126 23'D/S
3+00
2+00
GEOLOGIC PROFILE ALONG AXIS
SPILLWAY SECTION
Ground surface
Predominantly Silty Sand
LEGEND
Boring number
Water
CL Clay, low plasticity
CH Clay, high plasticity
SC Clayey sand
ML Silt, low plasticity
SM Silty sand
Core loss
Ground surface
Water table



ROFILE ALONG AXIS OF CONCRETE DAM



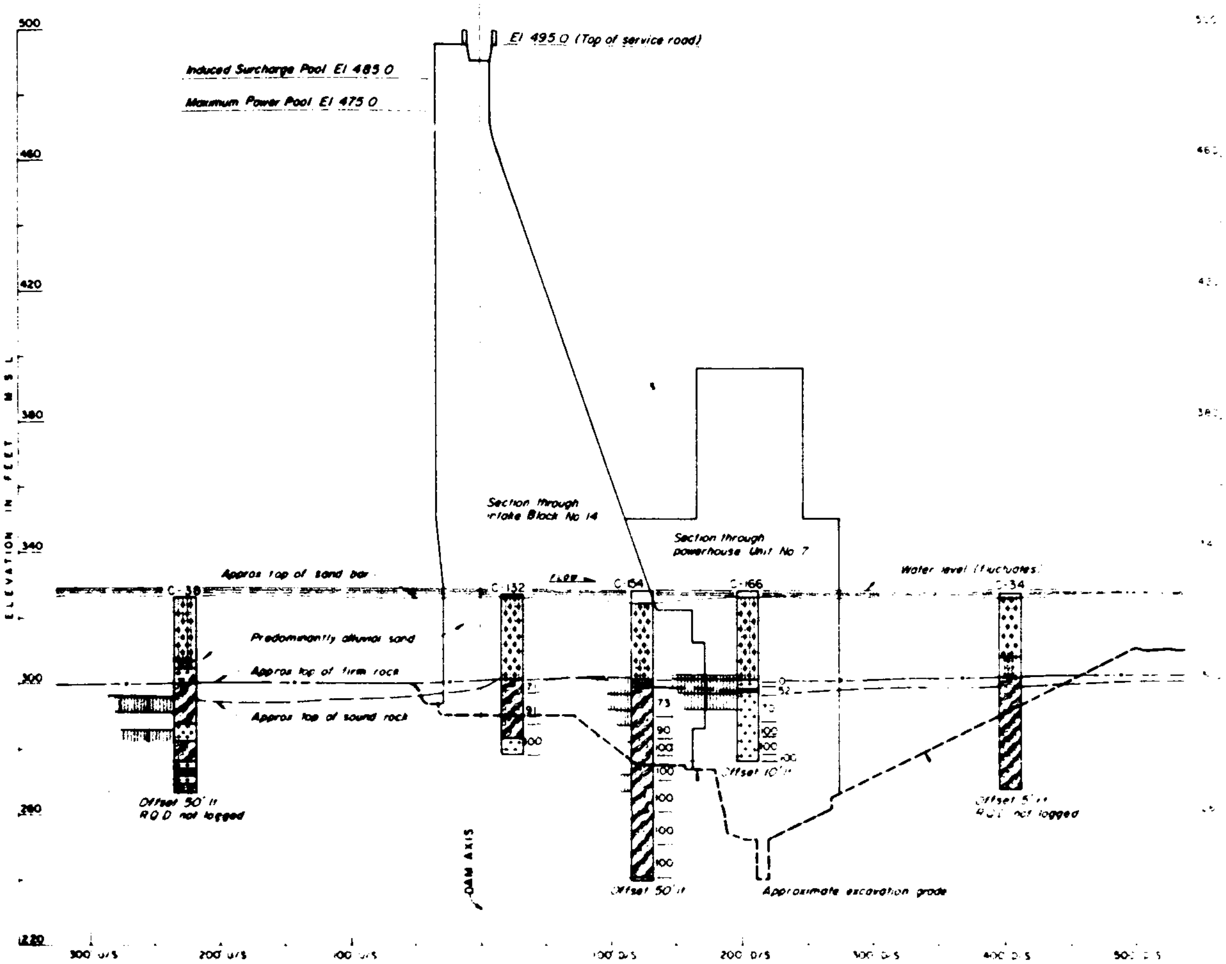
ROFILE ALONG AXIS OF CONCRETE DAM

NOTES

1. Top of dam El 495.0
2. Spillway design pool El 490.0
3. Flood control pool El 480.0
4. Maximum power pool El 475.0

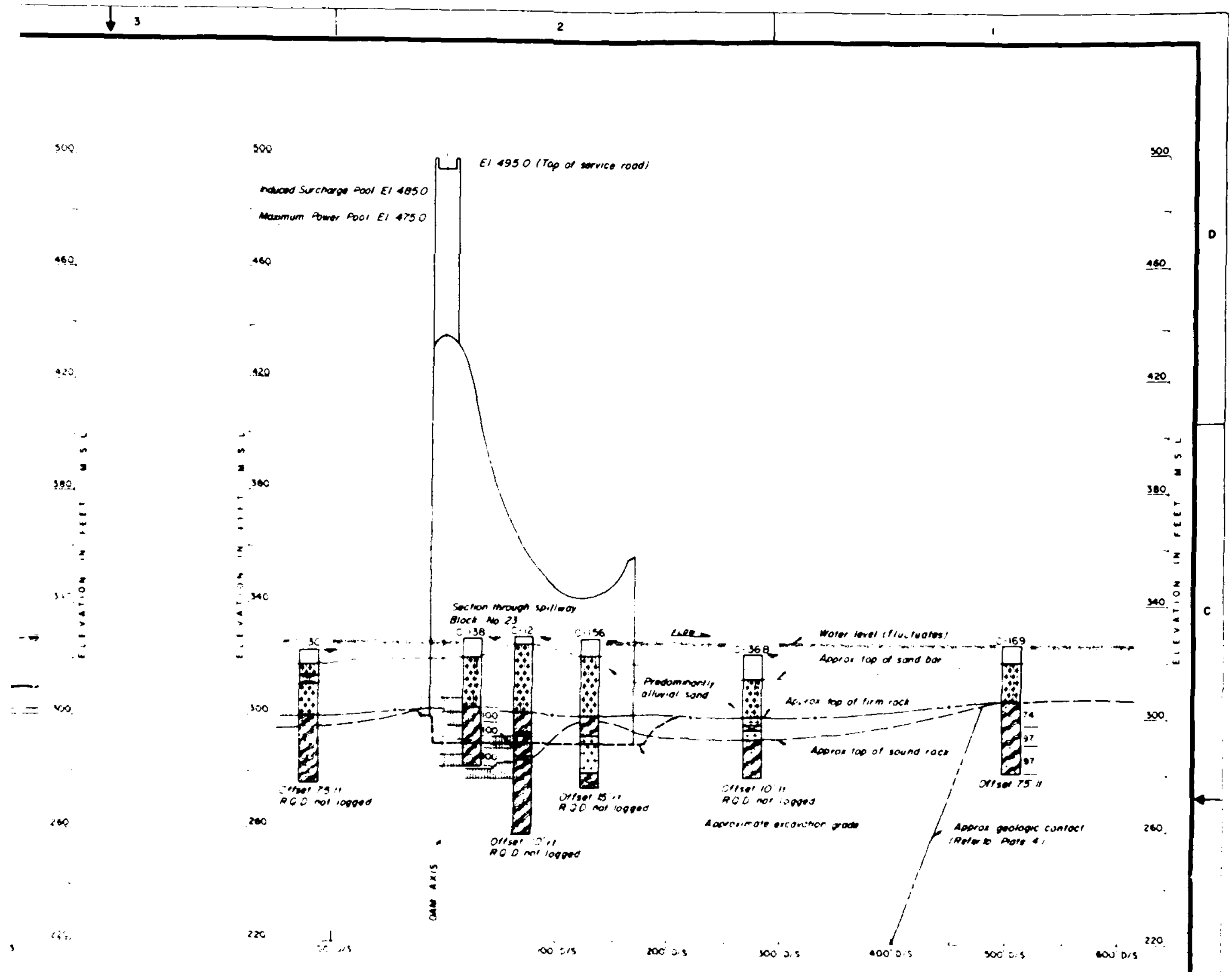
U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
GEOLOGIC PROFILE
ALONG AXIS OF CONCRETE DAM
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA



GEOLOGIC CROSS SECTION STA 8+10 (A-A)

NOTE: VERTICAL EXAGGERATION 2.5X
SCALE 1"=20' VERT
1"=50' HORIZ



GEOLOGIC CROSS SECTION STA 13+80 (B-B)

NOTES
1. For location of sections in plan view refer to Plate 12
2. Geologic logs of borings are maintained in the files of SASEN GG

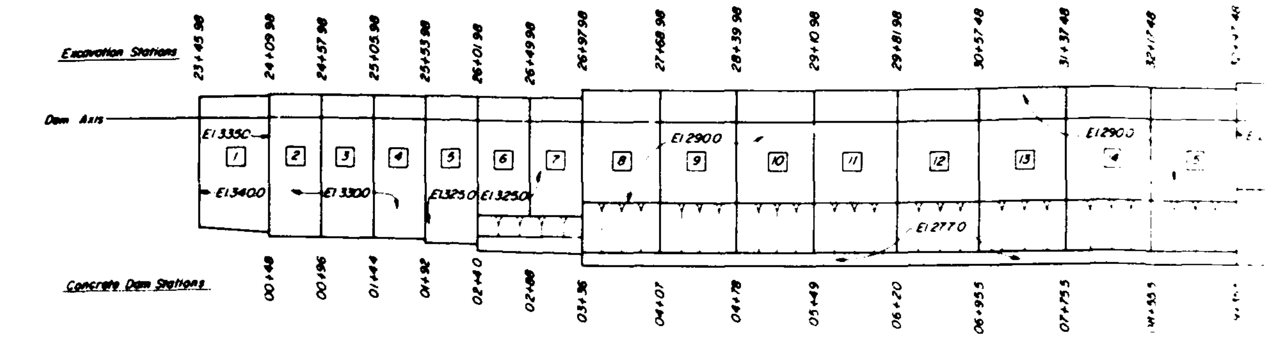
U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM

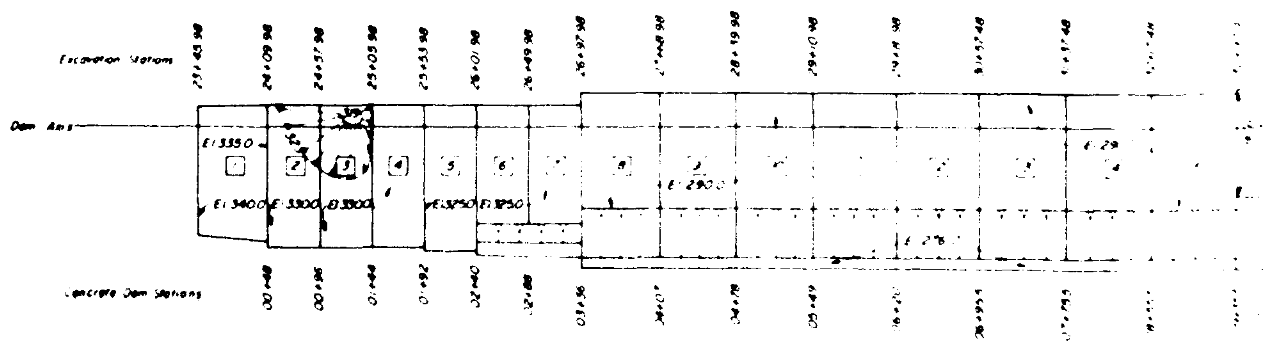
GEOLOGIC CROSS SECTIONS

RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER, GEORGIA AND SOUTH CAROLINA

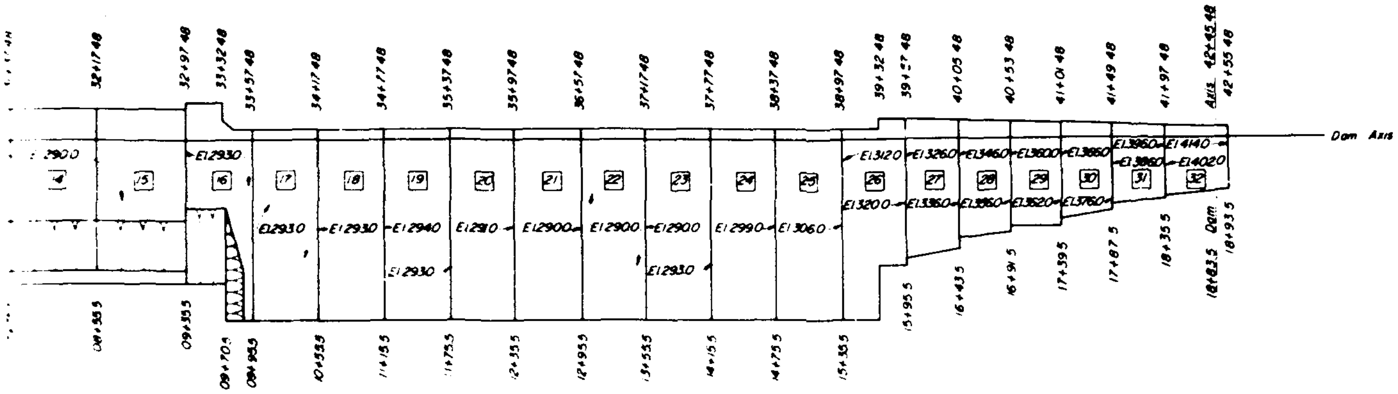
2



ORIGINAL CONTRA

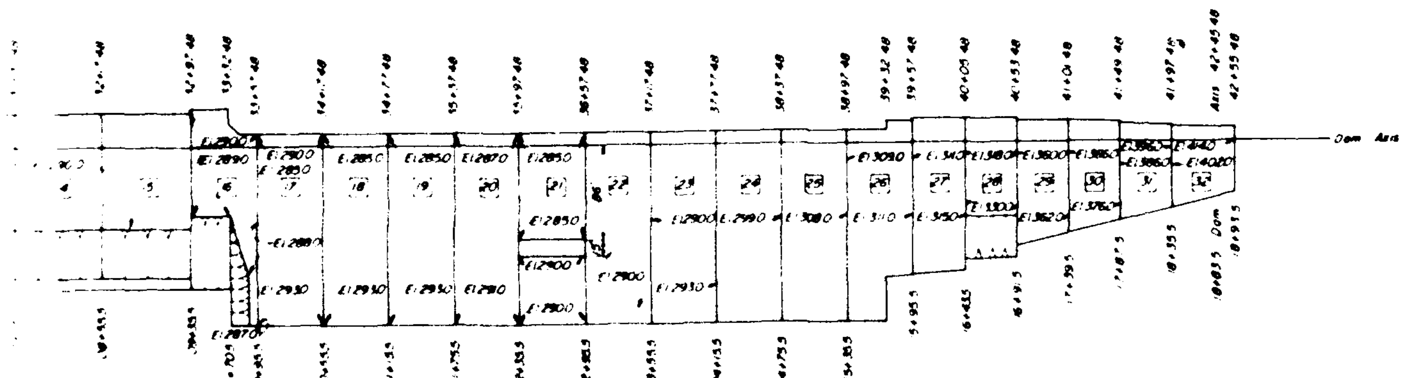


DIRECTED FOUNDATION CHANGES



ORIGINAL CONTRACT DESIGN

SCALE 1"=60'



FOUNDATION CHANGES ON EXCAVATION CONTRACT

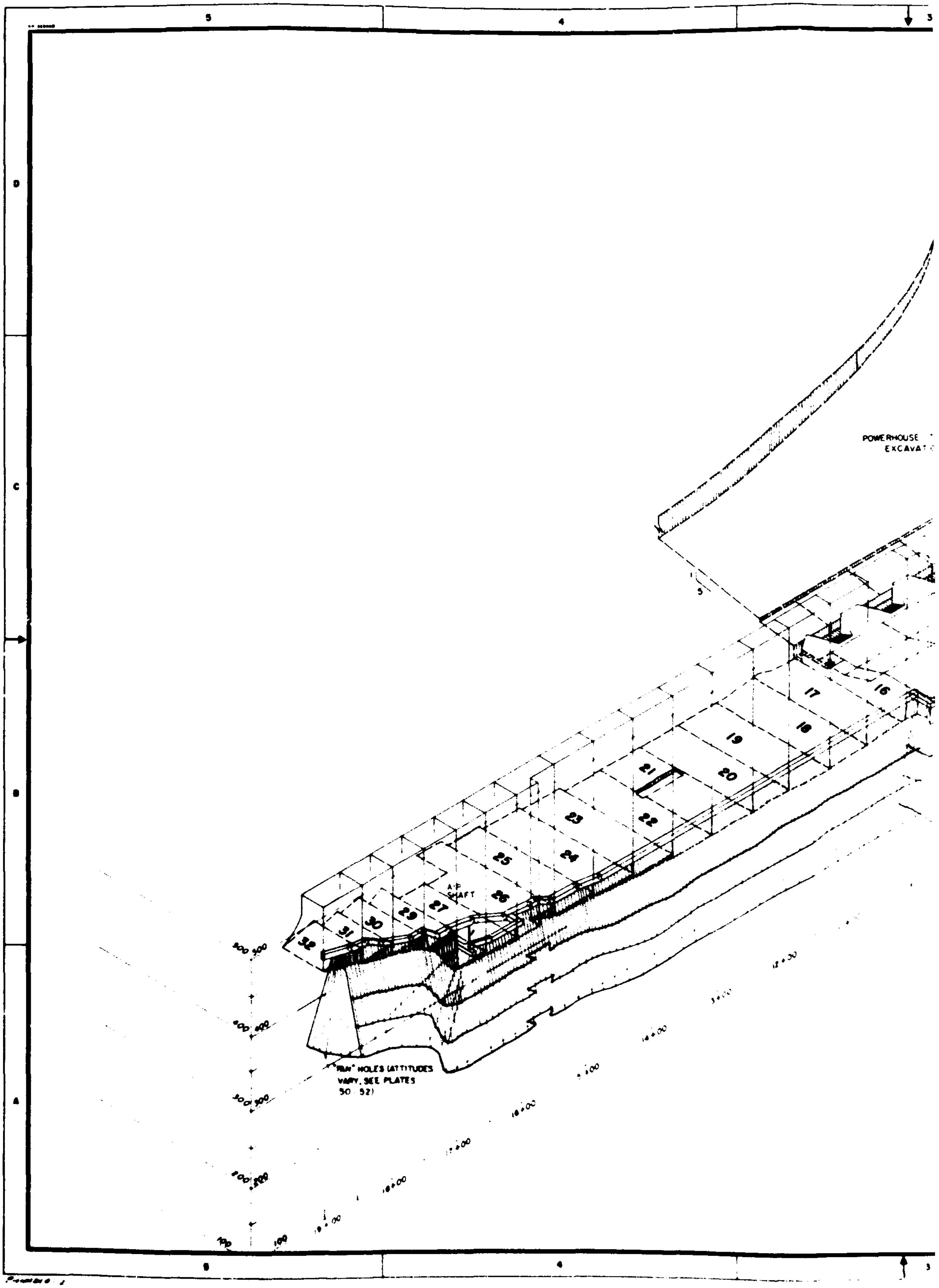
SCALE 1"=60'

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

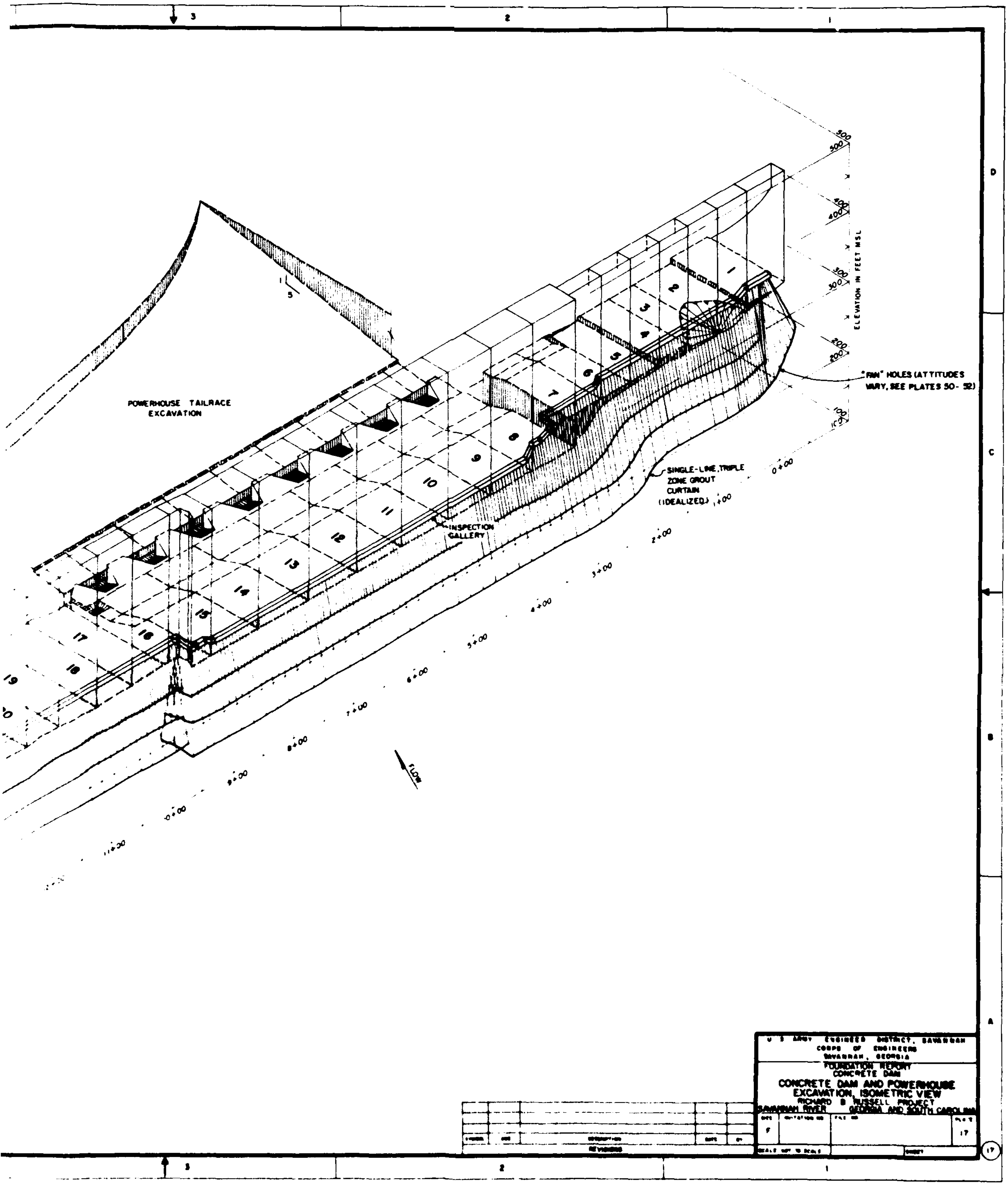
FOUNDATION REPORT
CONCRETE DAM
ORIGINAL AND CHANGED CONTRACT DESIGN

RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 16

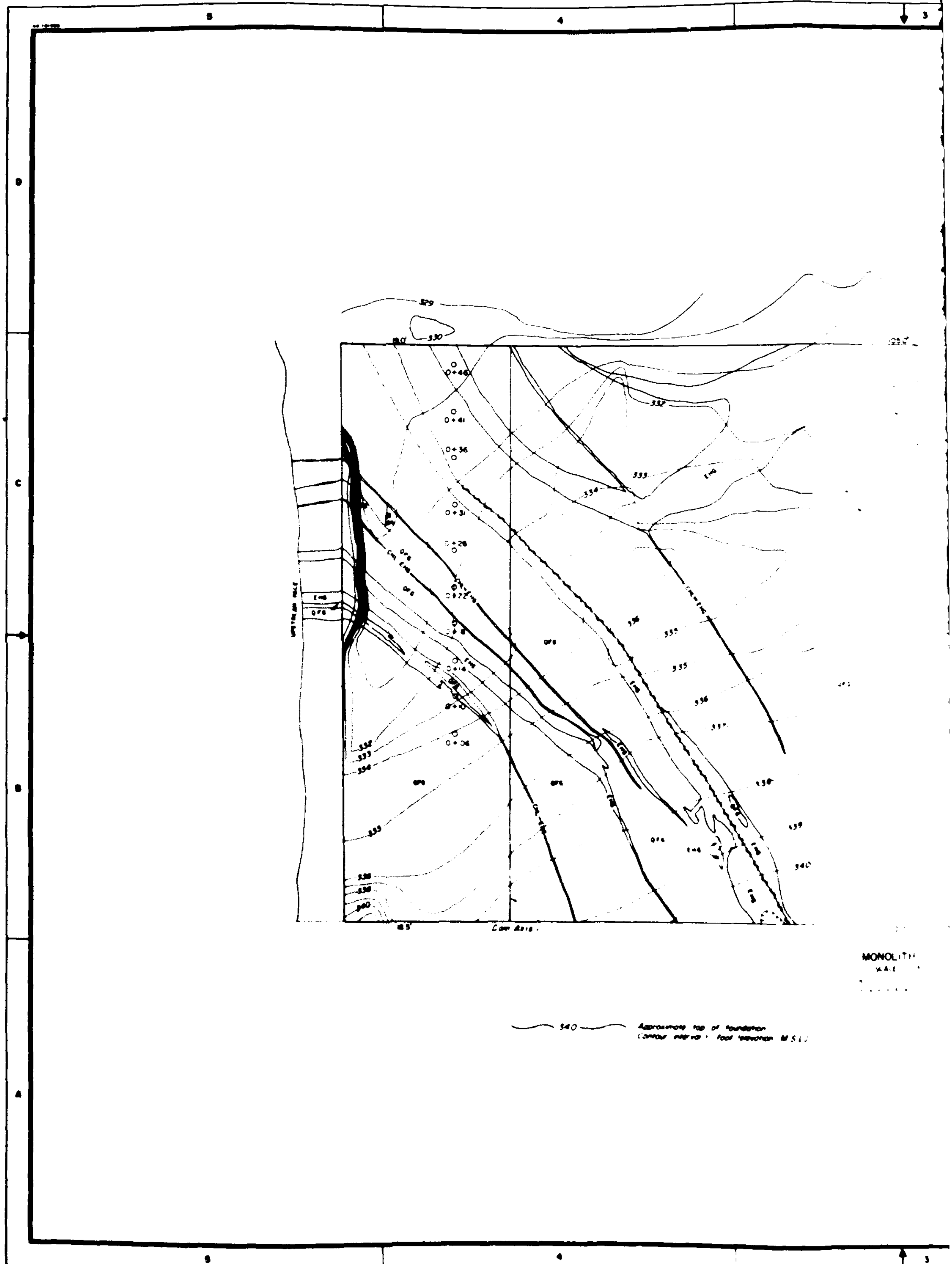


POWERHOUSE
EXCAVAT
A-F
SHAFT
"RAY" HOLES (ATTITUDES
VARY, SEE PLATES
50 - 52)
16
17
18
19
20
21
22
23
24
25
26
27
29
30
31
32



POWERHOUSE TAILRACE EXCAVATION
ELEVATION IN FEET MSL
500
400
300
200
100
"FAN" HOLES (ATTITUDES VARY, SEE PLATES 50- 52)
SINGLE-LINE, TRIPLE ZONE GROUT CURTAIN (IDEALIZED)
INSPECTION GALLERY
FLOW
0+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
CONCRETE DAM
CONCRETE DAM AND POWERHOUSE
EXCAVATION, ISOMETRIC VIEW
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
REVISIONS
17

2


329
330
332
333
334
335
336
337
338
339
340
Dam Axis
MONOLITH
540 Approximate top of foundation

MONOLITH I

SCALE 1:5

LEGEND

- QFP - Quartz feldspar porphyry
- EHG - Epidote hornblende gneiss mostly metamorphosed mafic dikes
- QFG - Quartz feldspar gneiss metamorphosed dacitic (volcanic) country rock
- Quartz (Schlieren) veins
- CHL EHG - Chloritic epidote hornblende gneiss
- Sheared rock zone
- Strike (in compass direction of straight line) and dip of measured joint
- Strike and dip of vertical joint
- Vertical fault
- Fault
- Slickensided area
- Entire area cut by very fine dikes that are too small to show at this scale of mapping
- • 79+00 Grout hole

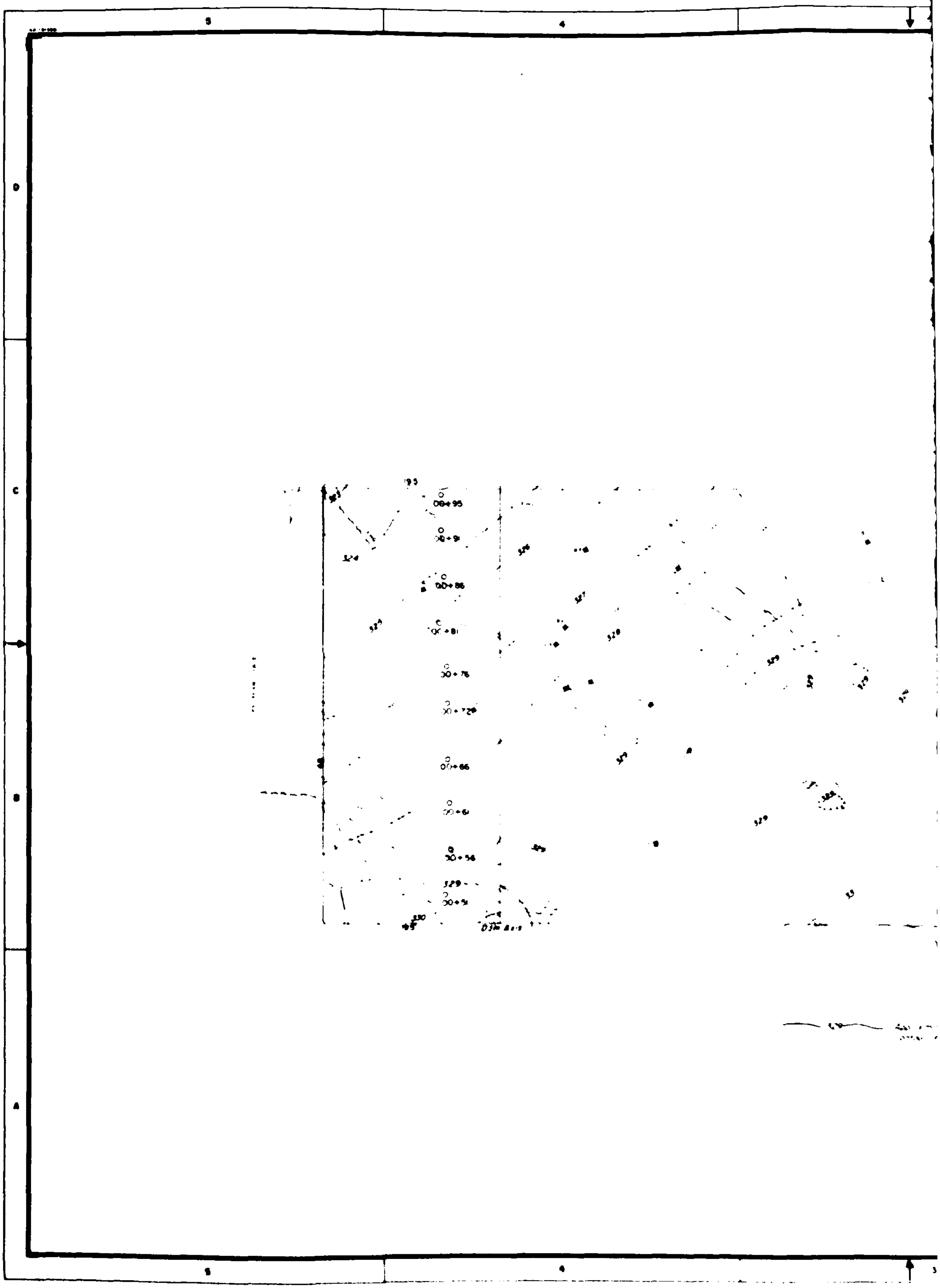
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
MONOLITH I
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

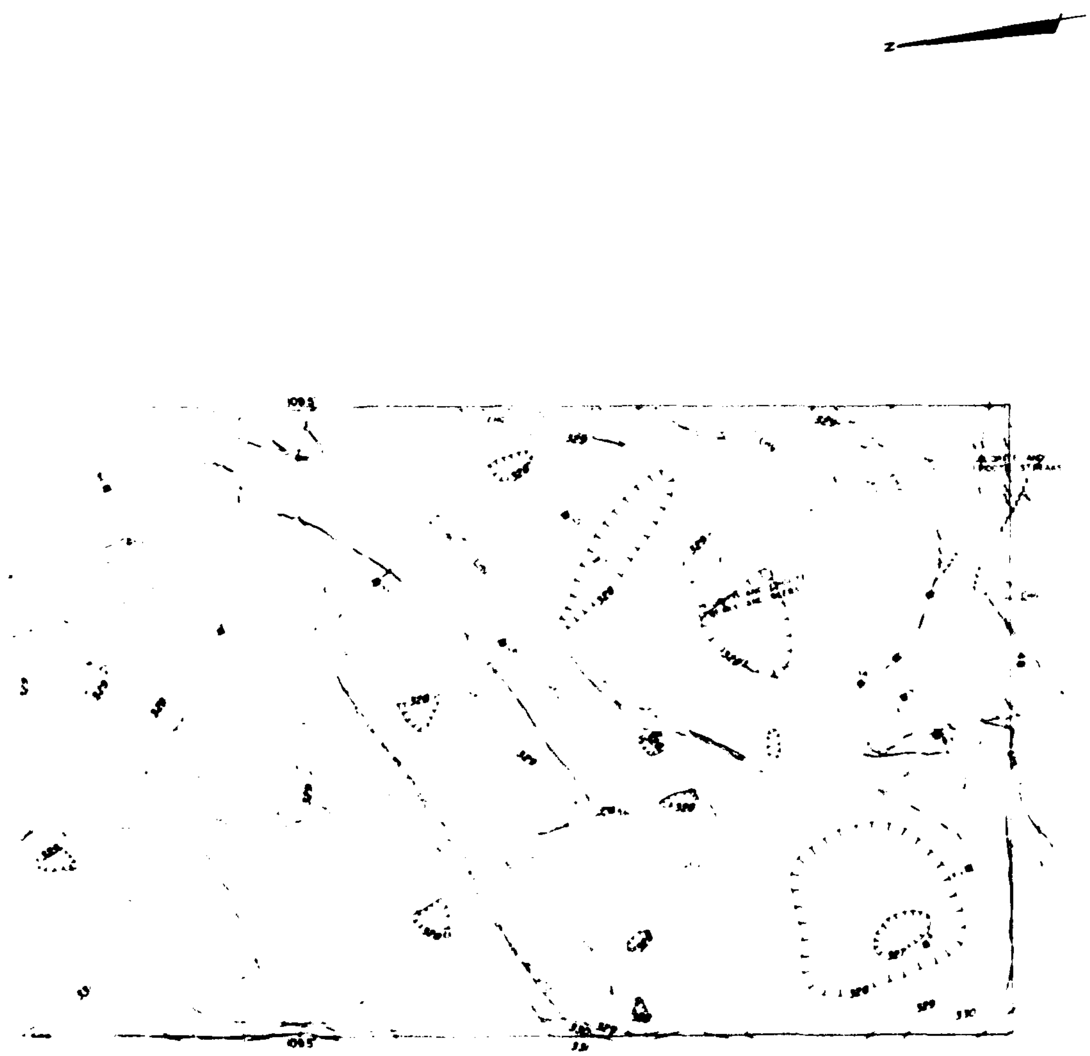
PLATE 18

SCALE AS SHOWN

2



5
4
D
C
B
A
00+95
00+91
00+86
00+81
00+76
00+72
00+66
00+61
00+56
00+51



MONOLITH 2

SCALE 1"=5'

329 Approximate top of foundation
Contour interval 1 foot (elevation MSL)

NOTE
For legend see Monolith 1

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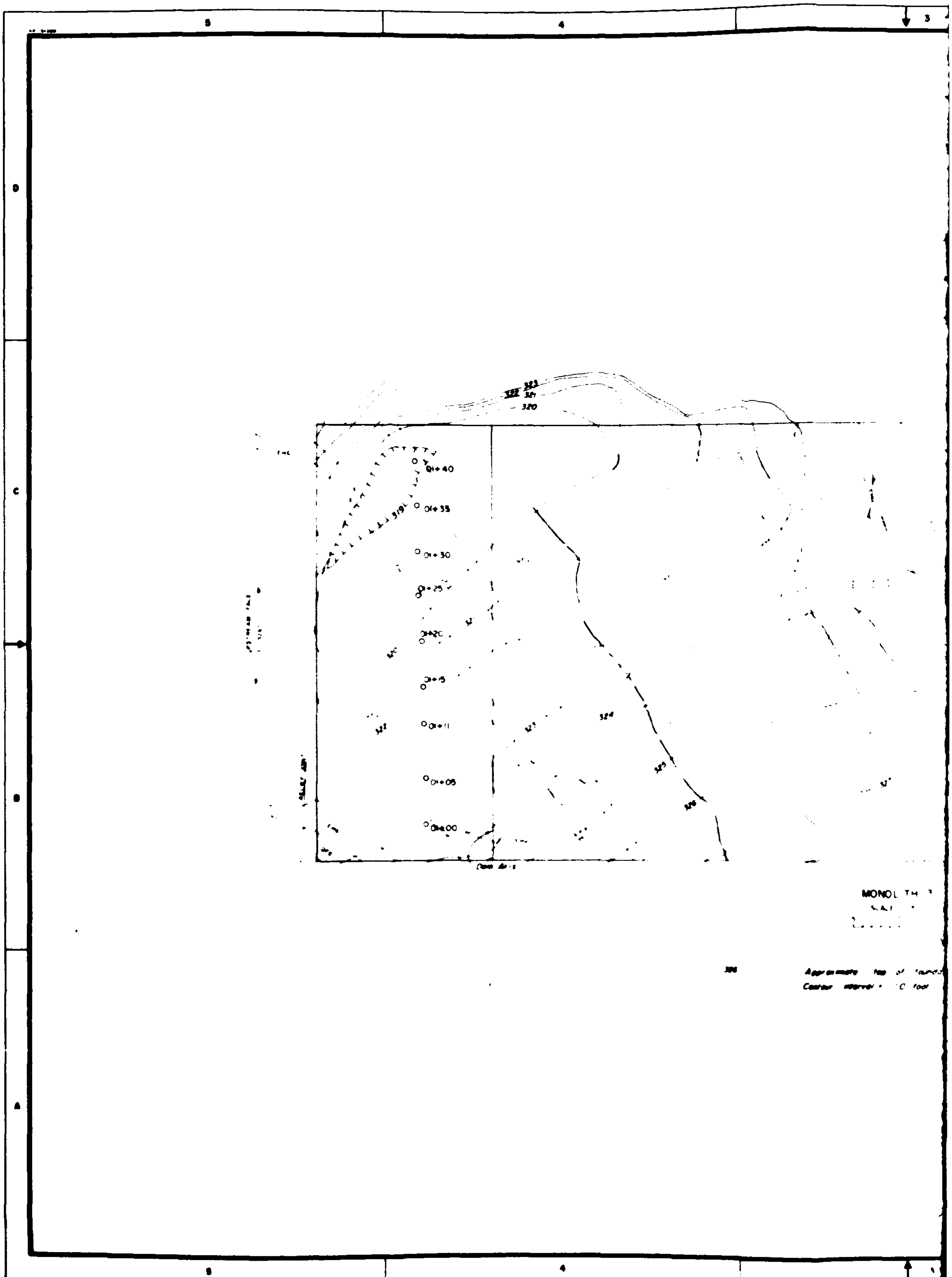
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

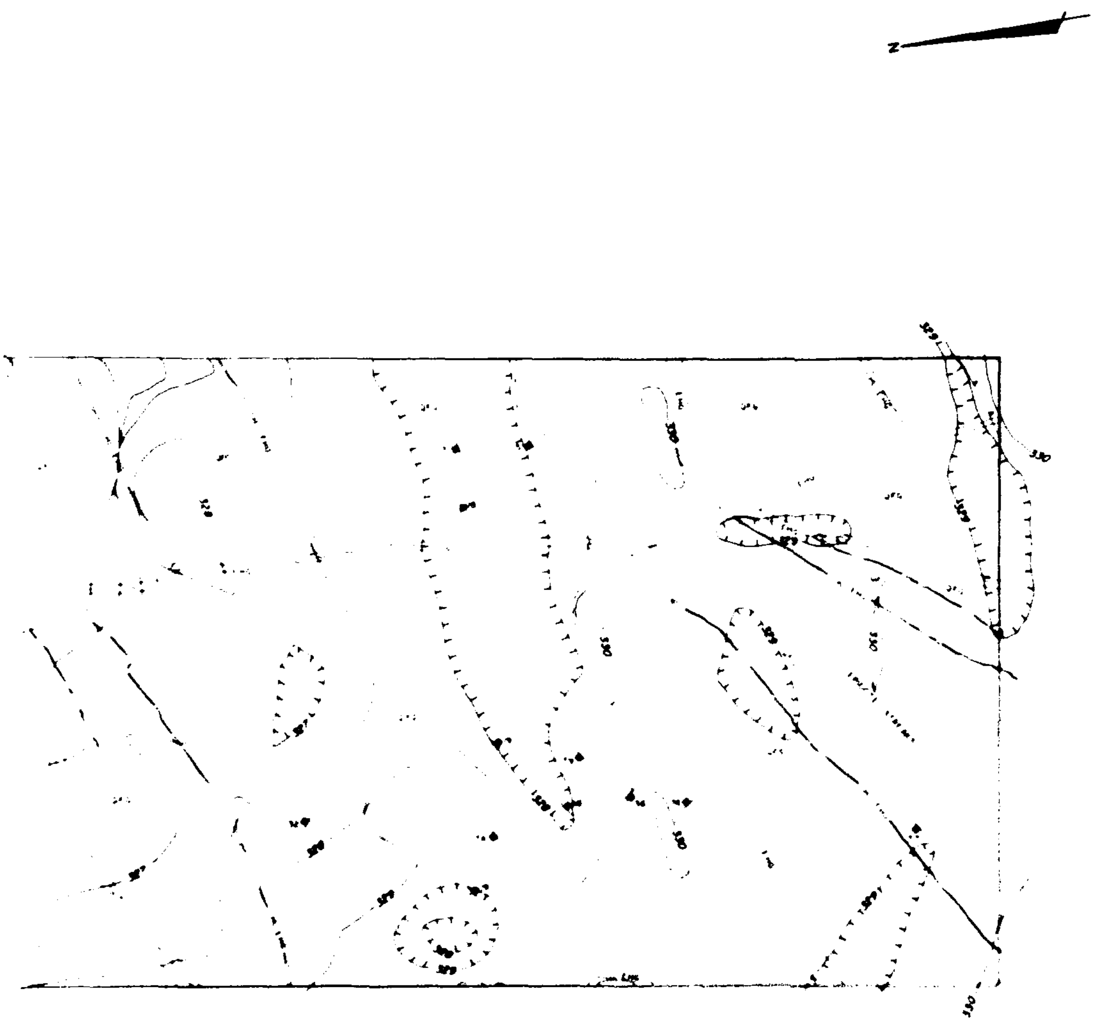
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 2
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 19

2



01+40
01+35
01+30
01+25
01+20
01+15
01+11
01+05
01+00
320
321
323
319



MONOLITH 3

SCALE 1"=5'

NOTE: For legend see Monolith 1

Approximate top of foundation

Contour interval = 10 feet (elevation M.S.L.)

U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT

CONCRETE DAM

MONOLITH 3

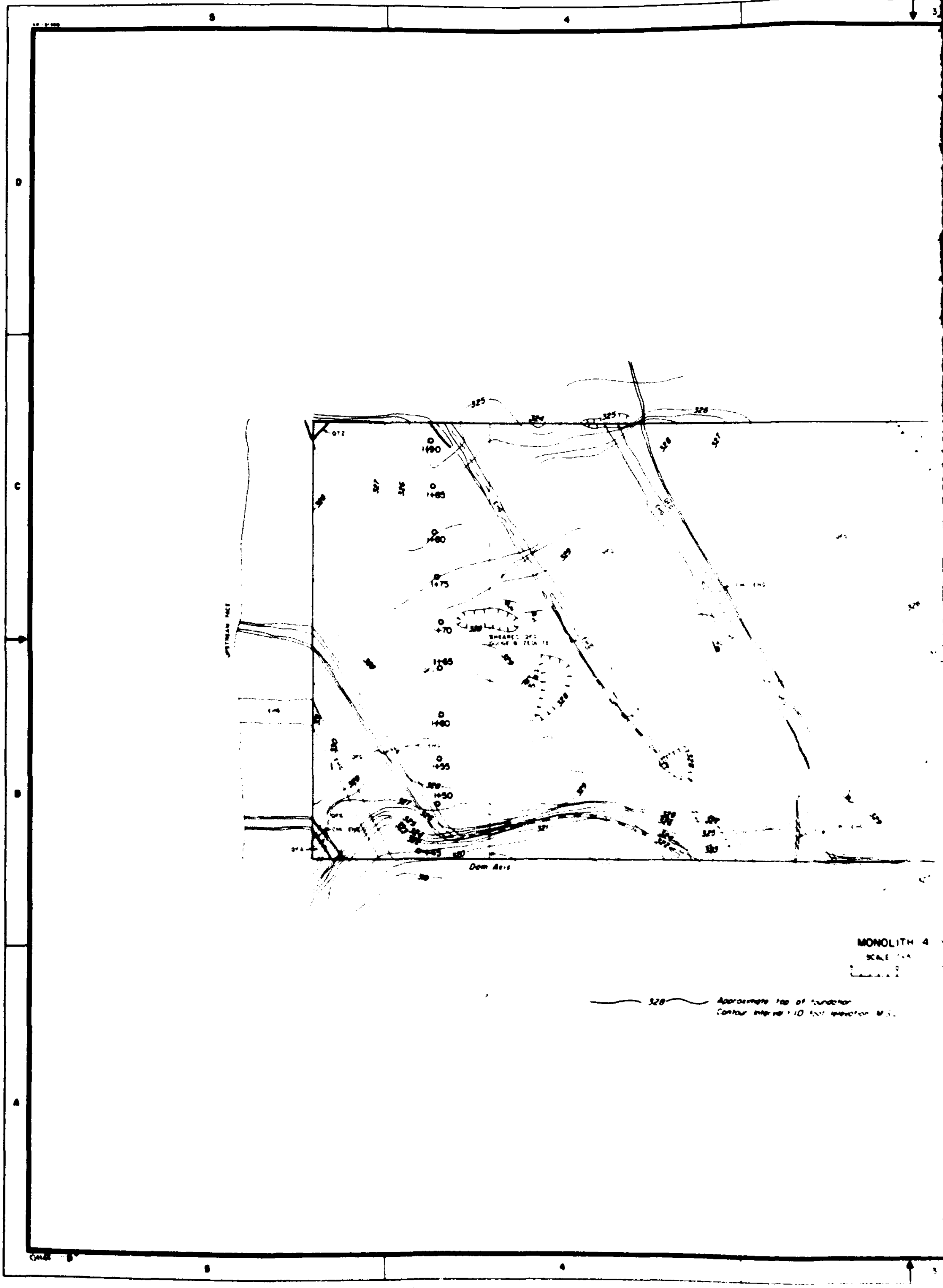
RICHARD B. RUSSELL PROJECT

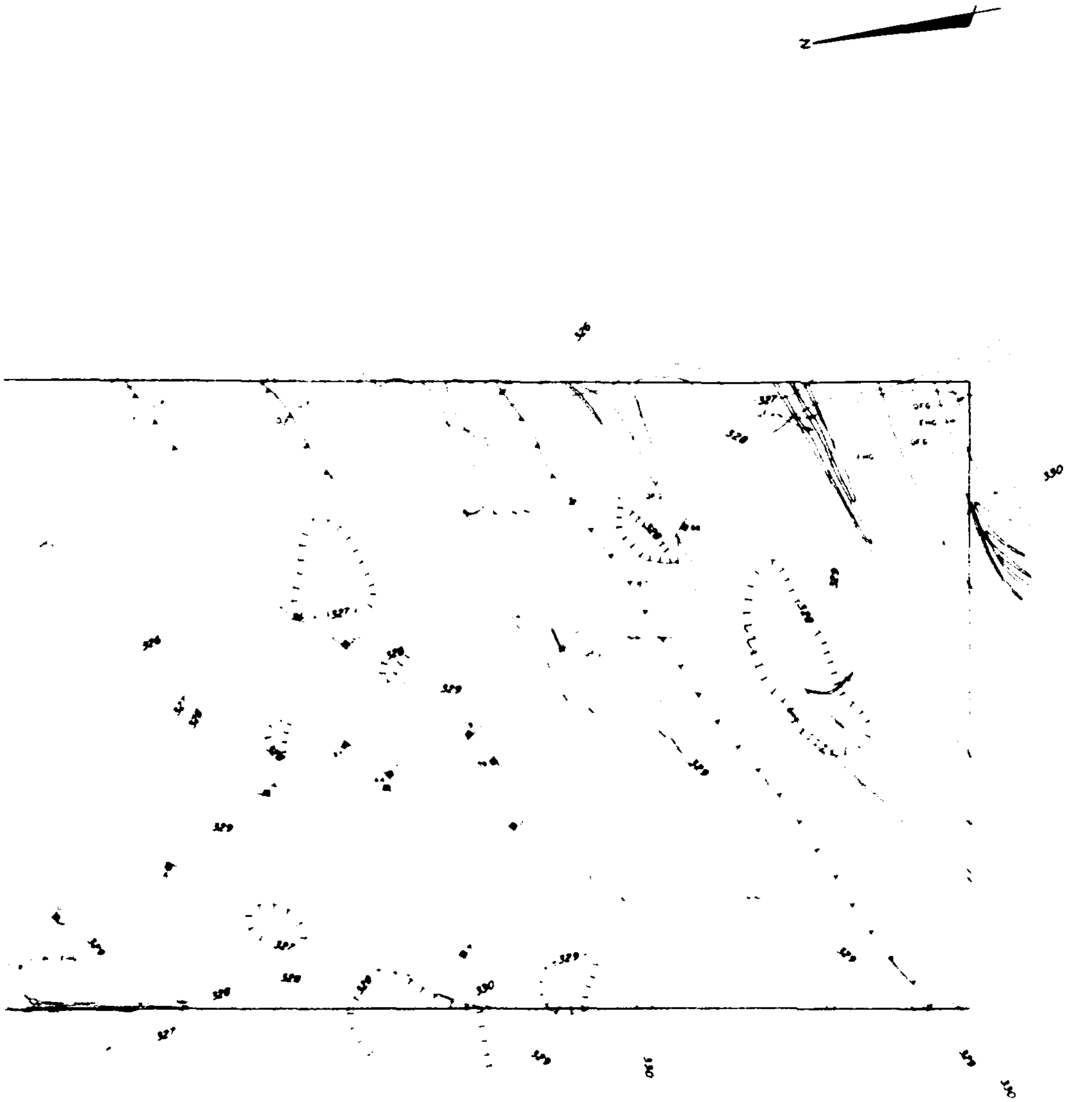
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE	INVITATION NO.	DRAWING NO.	PLATE
F			20

SCALE AS SHOWN

SYMBOL	ZONE	DESCRIPTION	DATE	BY
		REVISIONS		


MONOLITH 4
Dam Axis
Approximate top of foundation



MONOLITH 4

SCALE 1"=5'

Foundation
rock elevation M.S.L.

NOTE
For legend see Monolith 1

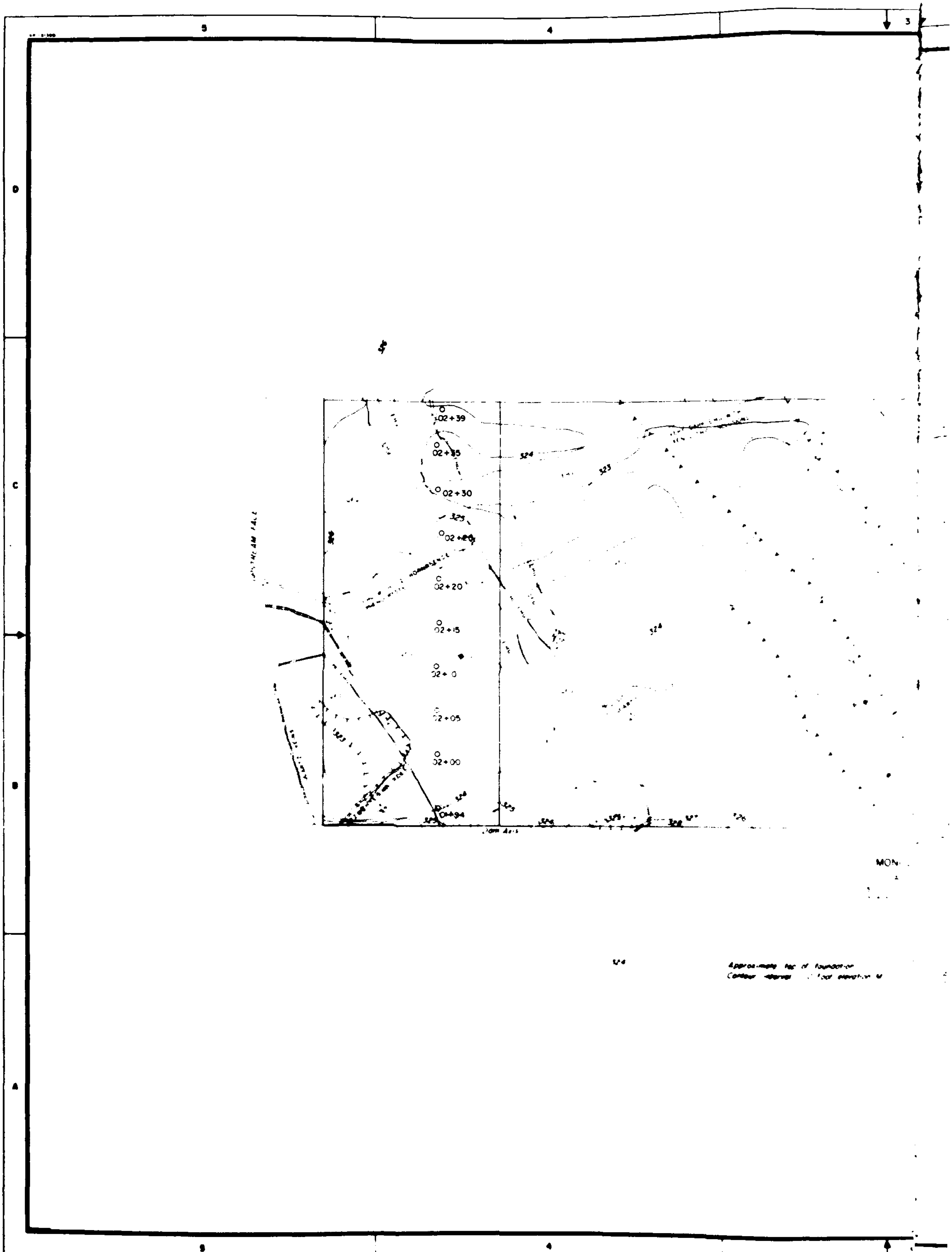
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

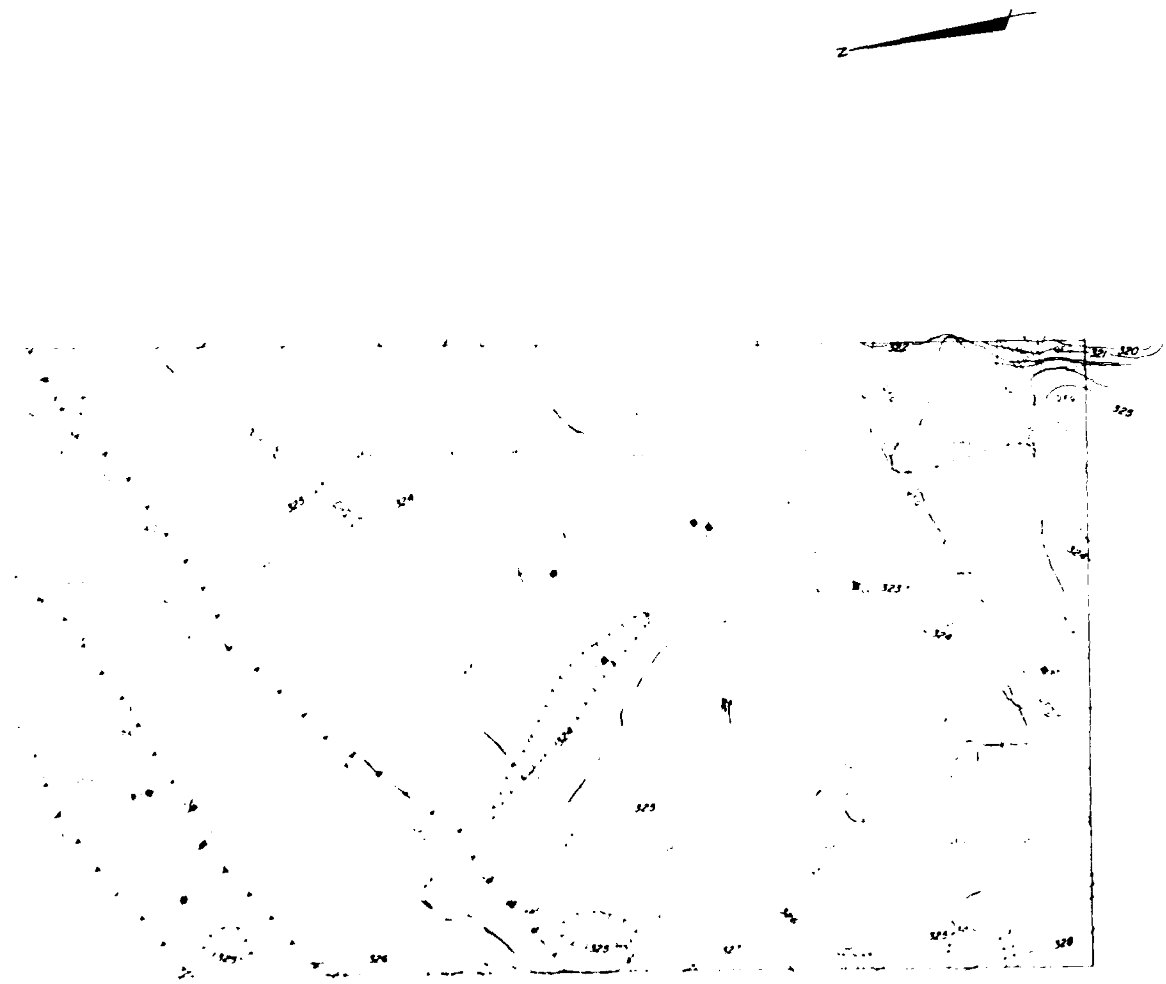
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 4
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 21

REVISIONS

2


5
4
3
D
C
B
A
02+39
02+35
02+30
02+20
02+15
02+05
02+00
324
323
325
MON



MONOLITH 5

SCALE 1:5

NOTE

For legend see Monolith 1

...the top of foundation
...interval - 1.0 foot (elevation M.S.L.)

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SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
MONOLITH 5
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

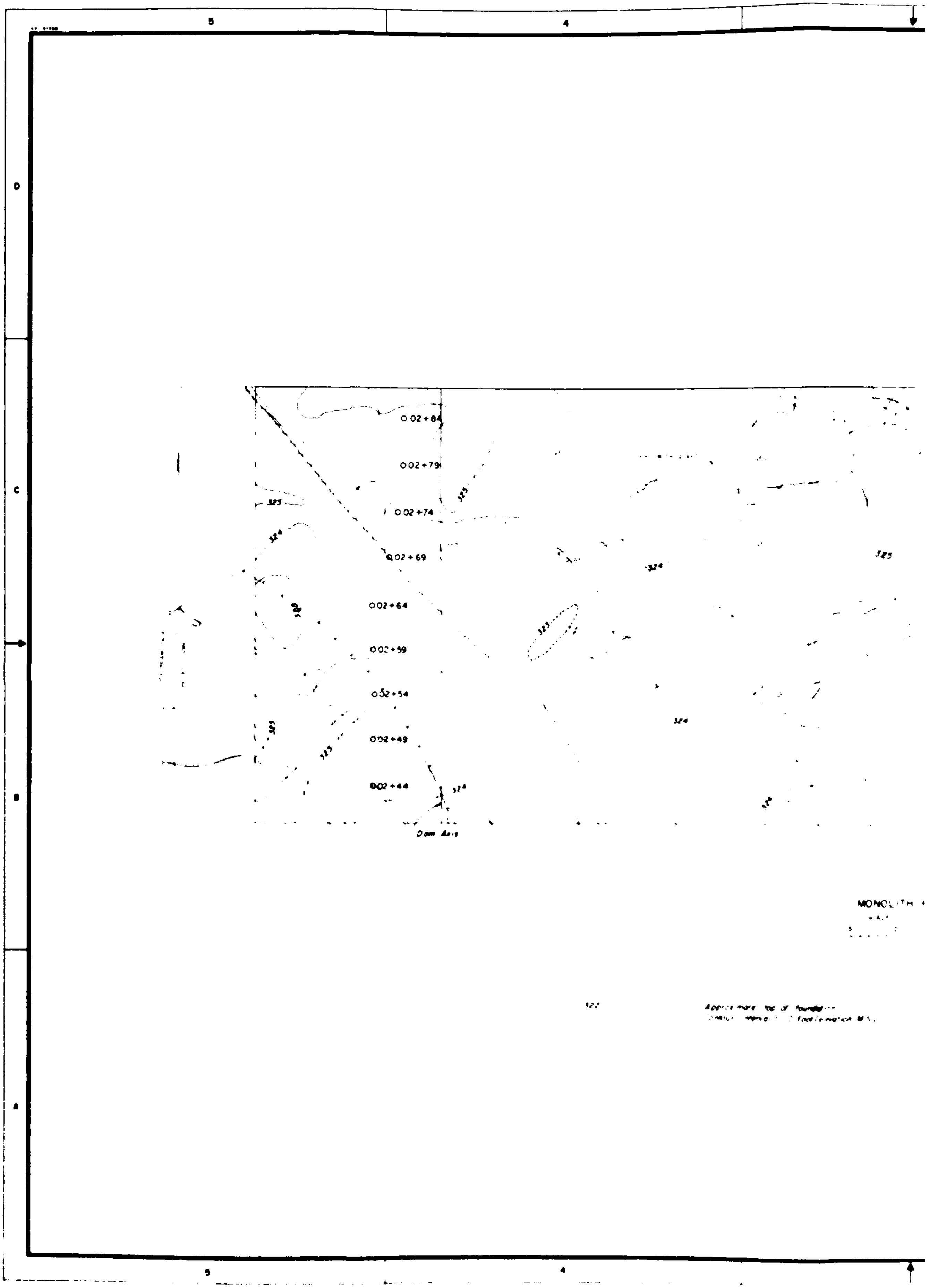
SIZE			PLATE
F			22

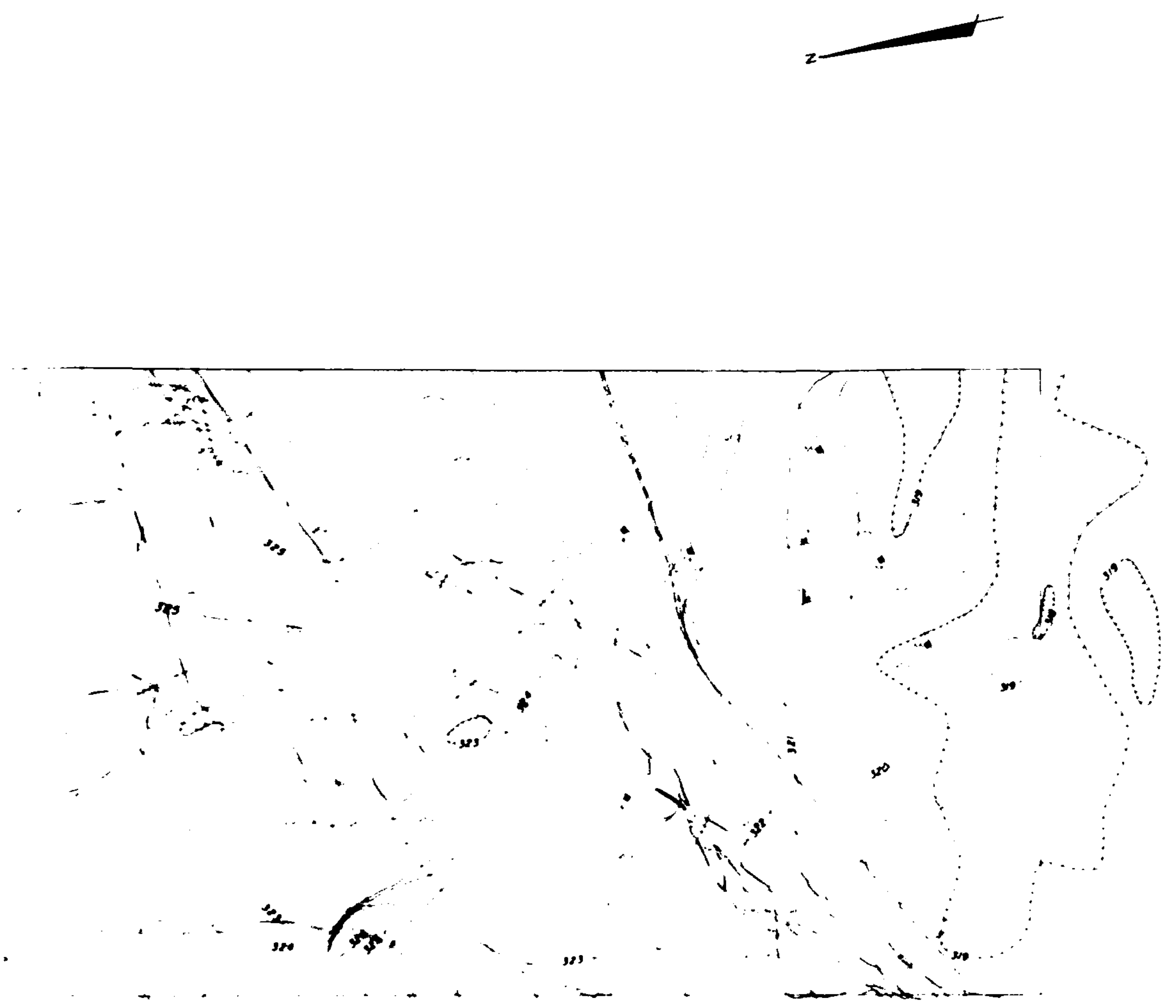
SCALE AS SHOWN SHEET

REVISIONS

22

2


5
4
D
C
B
A
0 02+84
0 02+79
0 02+74
0 02+69
0 02+64
0 02+59
0 02+54
0 02+49
0 02+44
325
324
Dam Axis



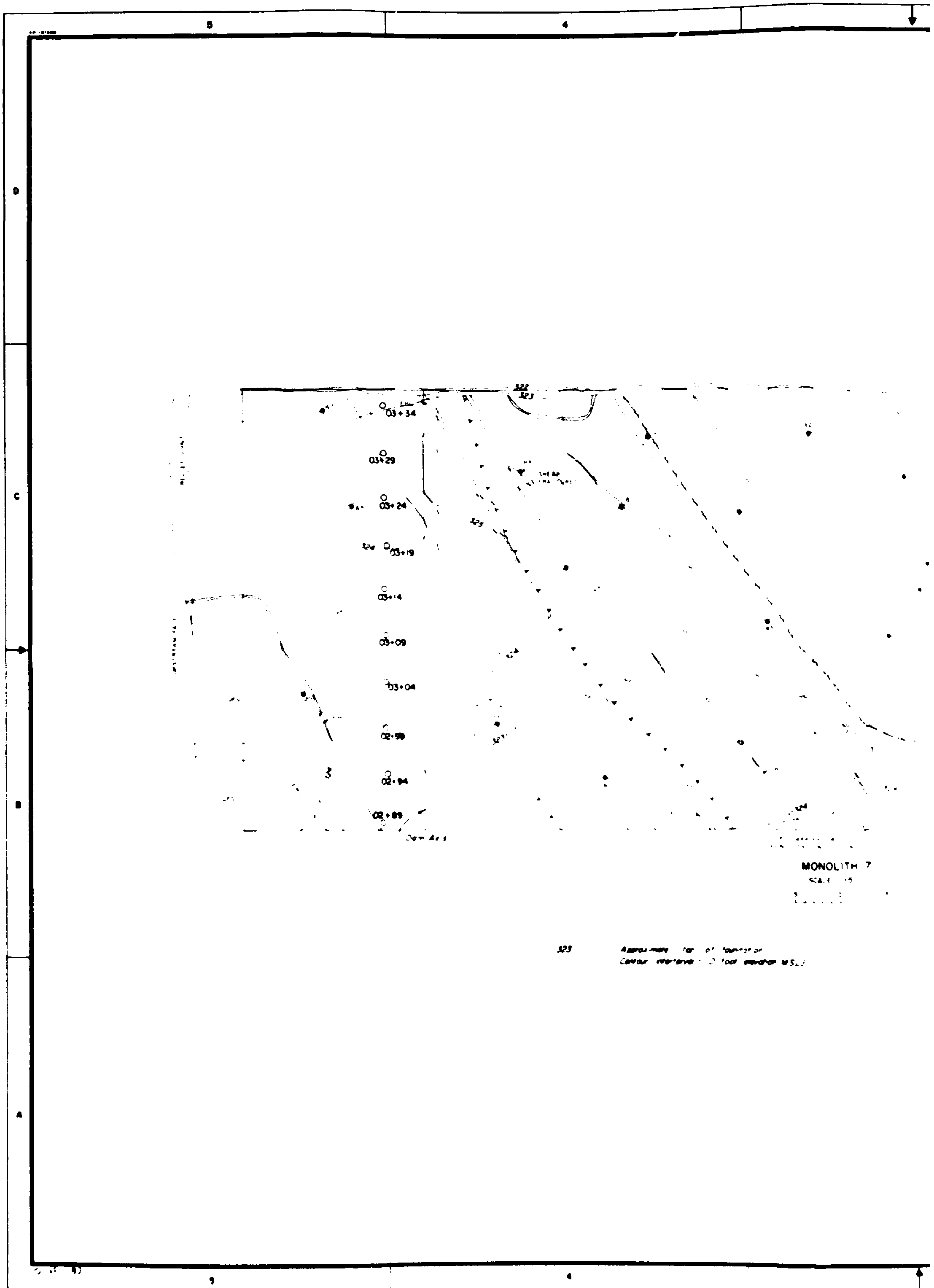
MONOLITH 6
SCALE 1:5

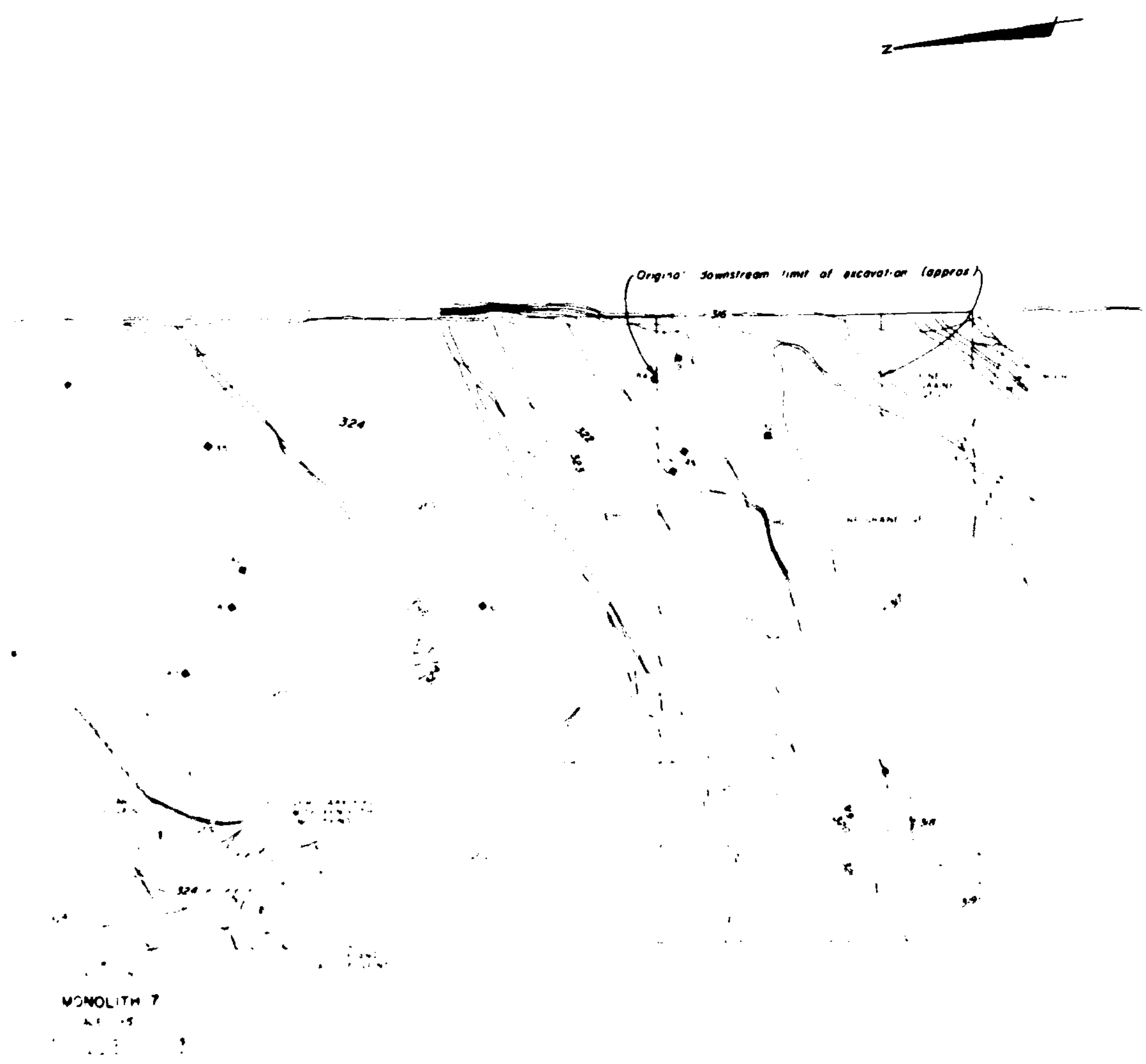
NOTE
For legend see Monolith 1

U. S. ARMY ENGINEER DISTRICT, SAVANNAH CORPS OF ENGINEERS SAVANNAH, GEORGIA
FOUNDATION REPORT CONCRETE DAM MONOLITH 6 RICHARD B. RUSSELL PROJECT SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
PLATE 23

23

2


03+34
03+29
03+24
03+19
03+14
03+09
03+04
02+99
02+94
02+89
322
323
Dam Axis
MONOLITH 7
323

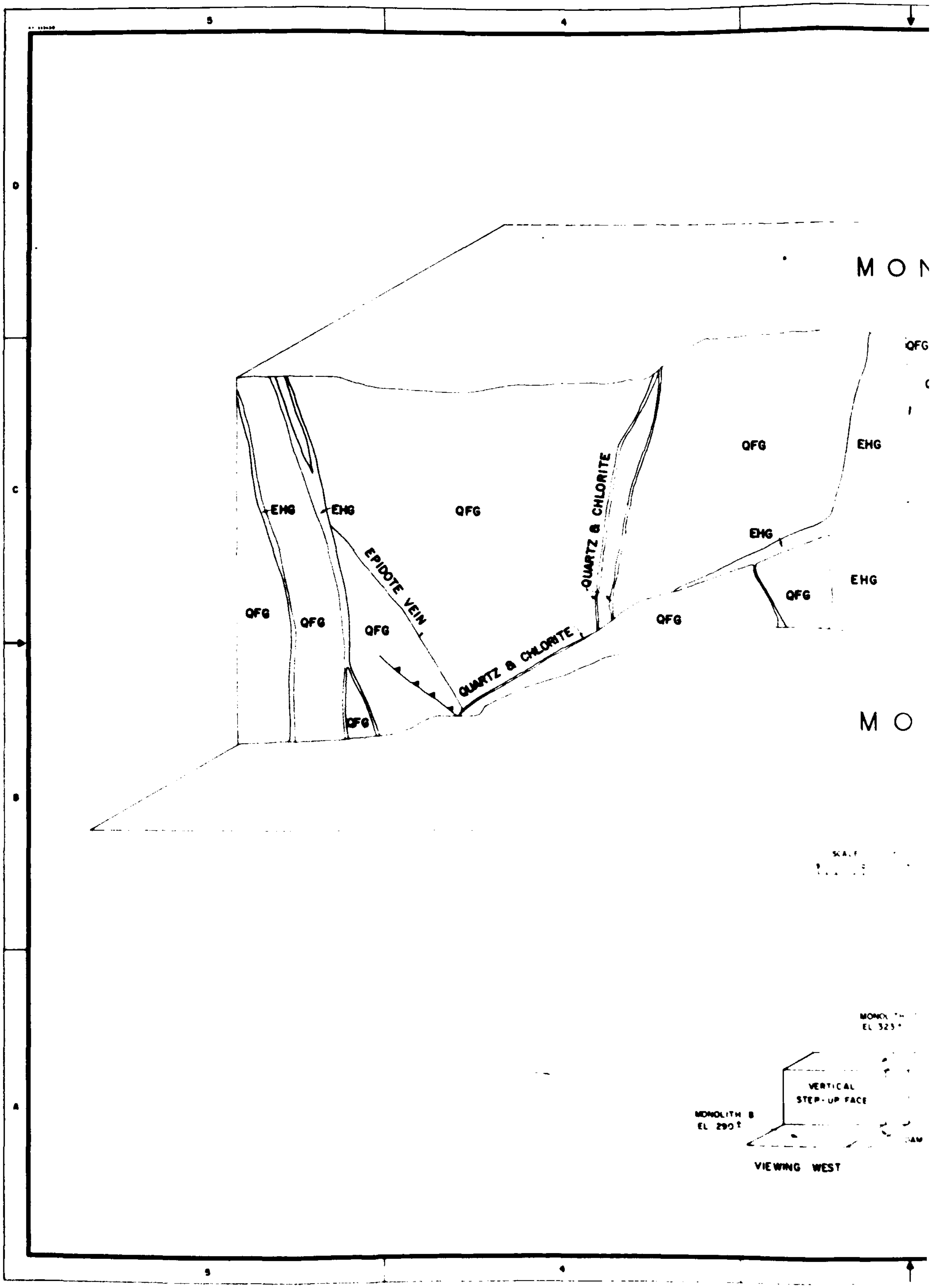


NOTE For legend see Monolith

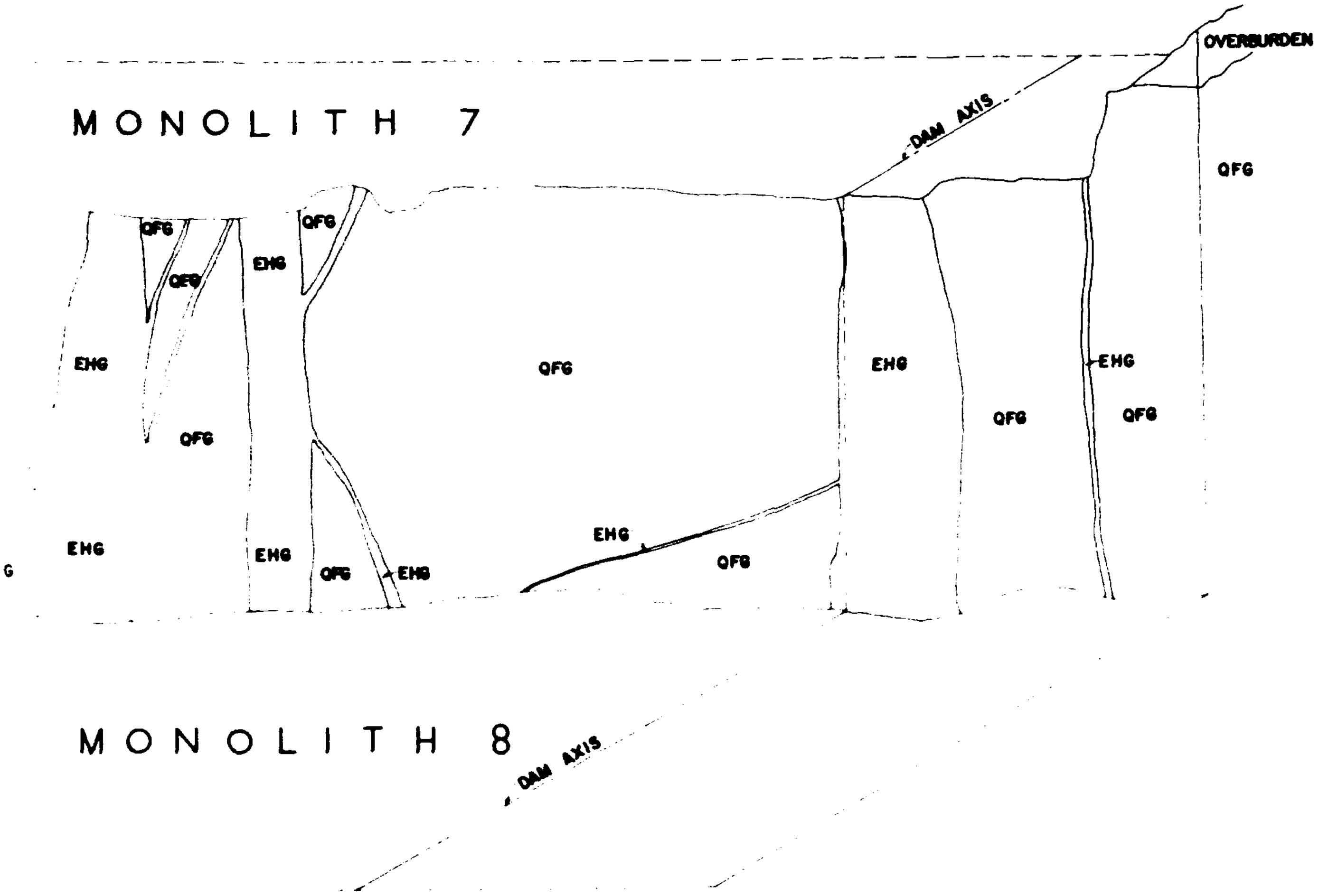
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
MONOLITH 7
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

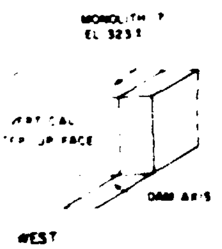
PLATE 24



QFG
EHG
EPIDOTE VEIN
QUARTZ & CHLORITE
QUARTZ & CHLORITE
VERTICAL STEP-UP FACE
MONOLITH 8
EL 290±
VIEWING WEST

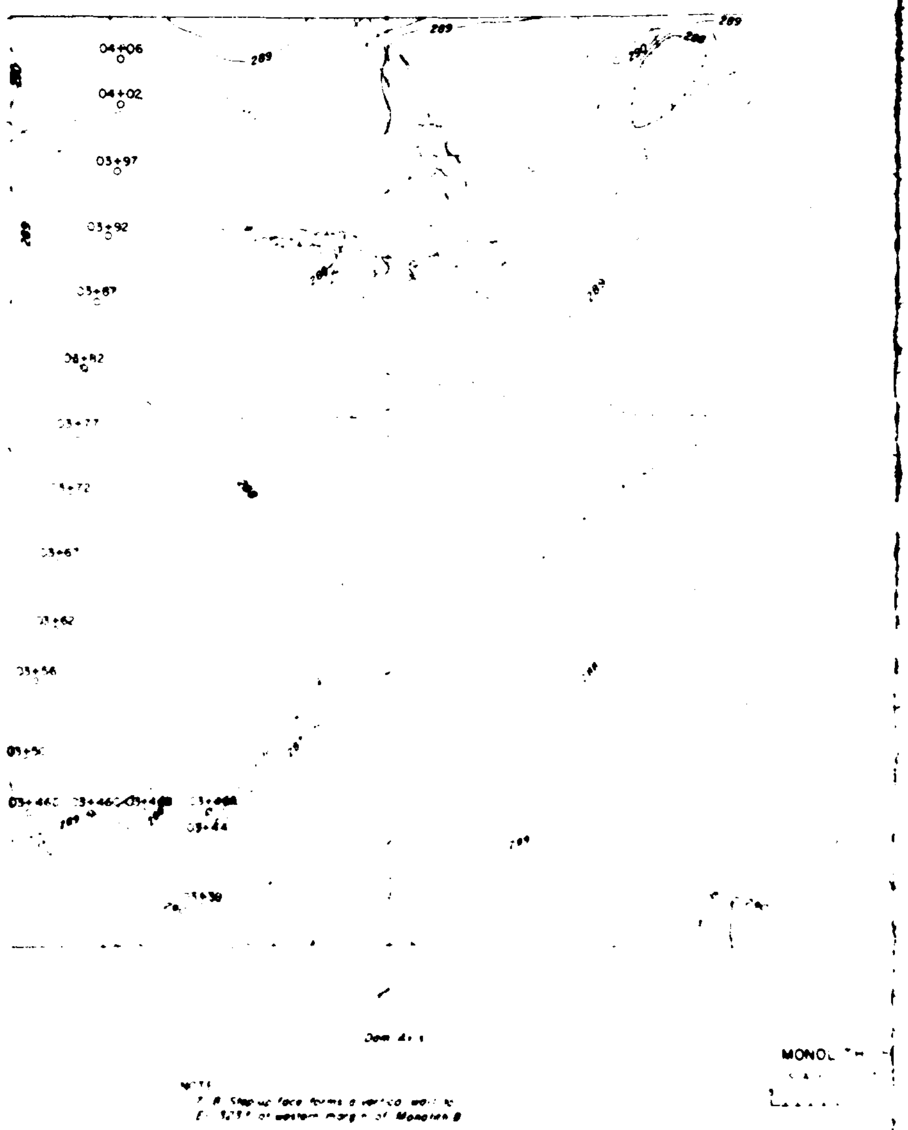


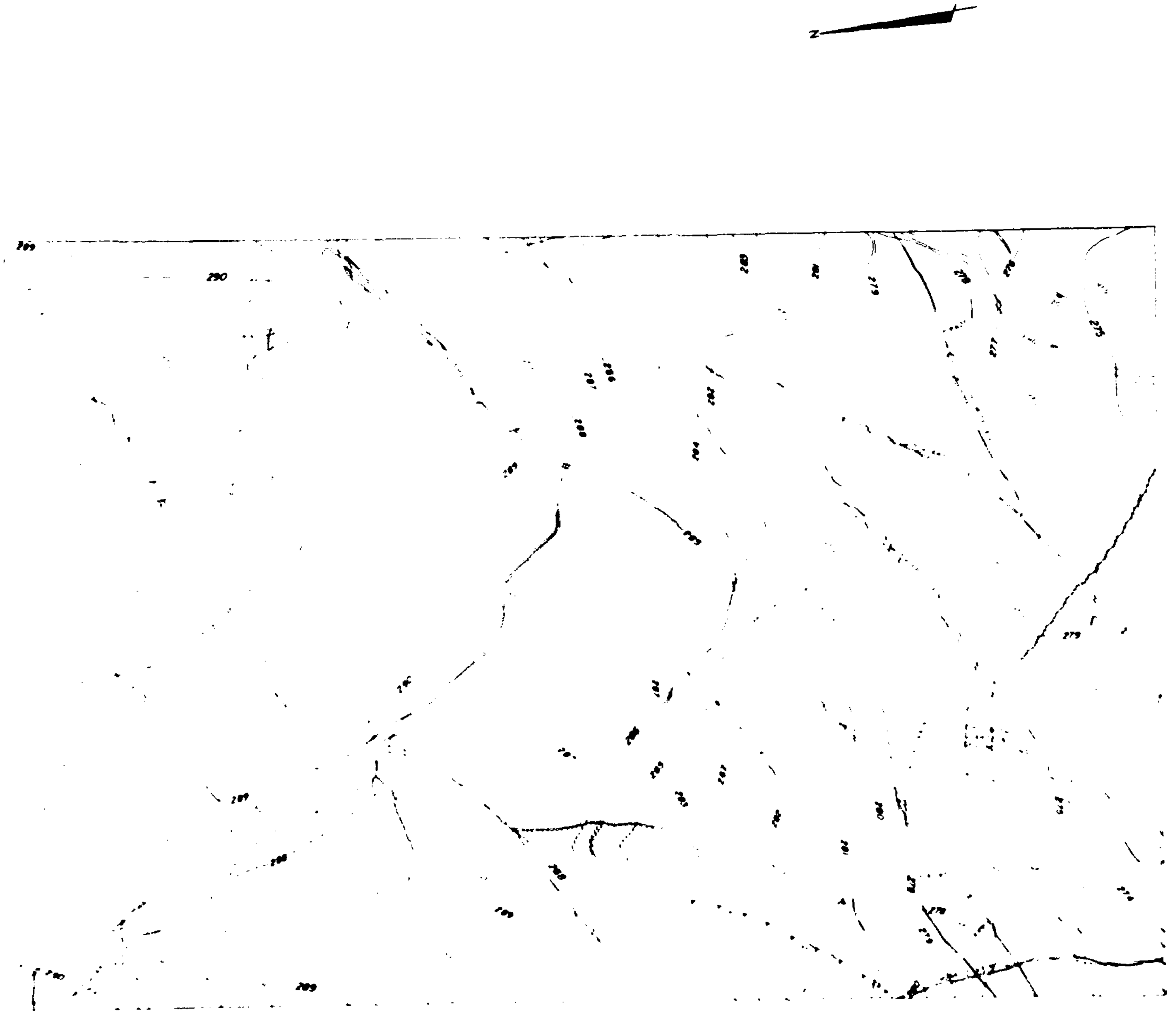
NOTE
FOR LEGEND SEE MONOLITH 1



U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
CONCRETE DAM
GEOLOGY OF VERTICAL STEP-UP FACE
BETWEEN MONOLITHS 7 & 8
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 25


04+06
04+02
03+97
03+92
289
MONOL



MONOLITH 8

289 Approximate top of foundation
Contour interval 1.0 foot (elevation M.S.L.)

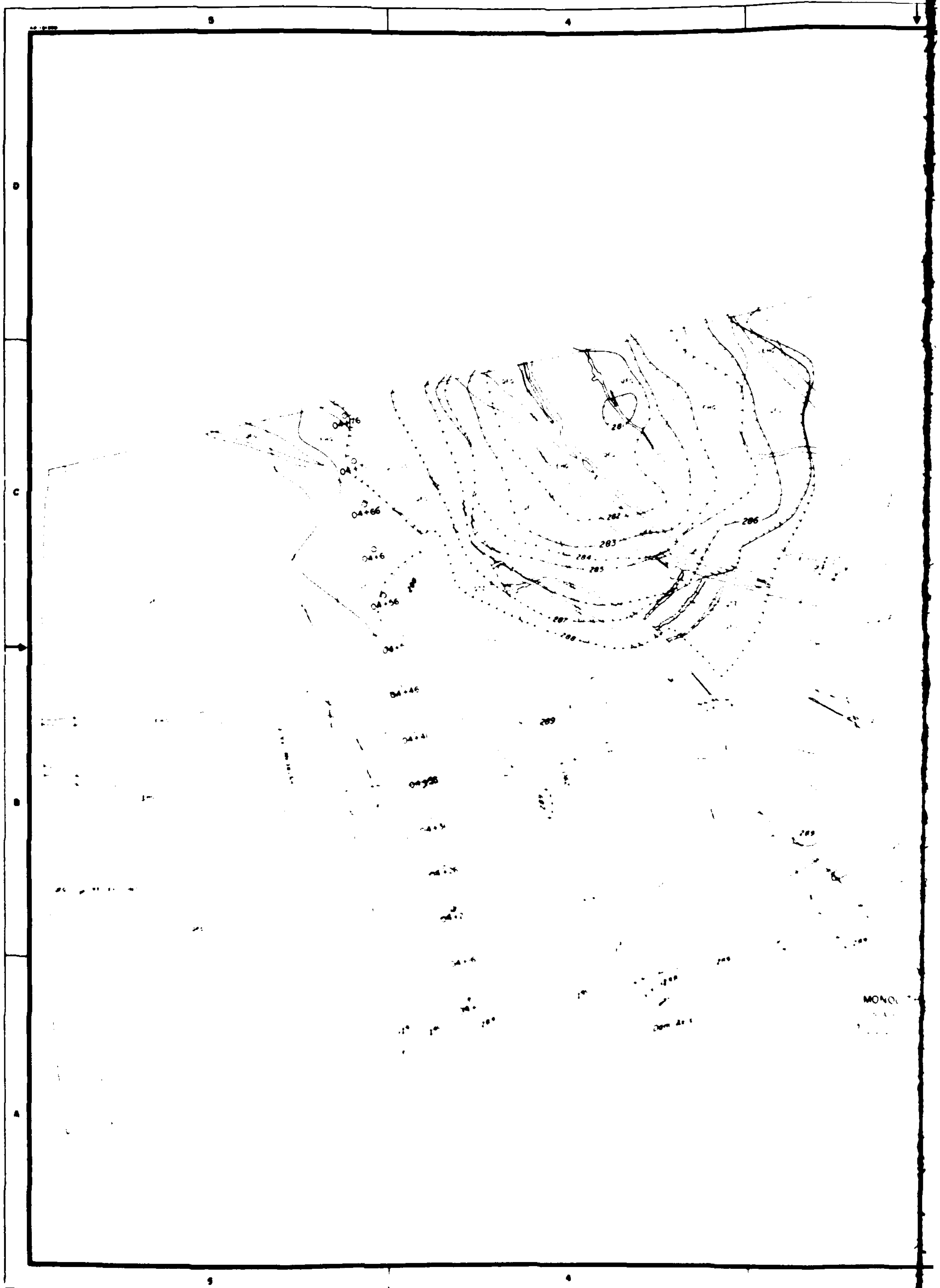
For legend see Monolith

U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

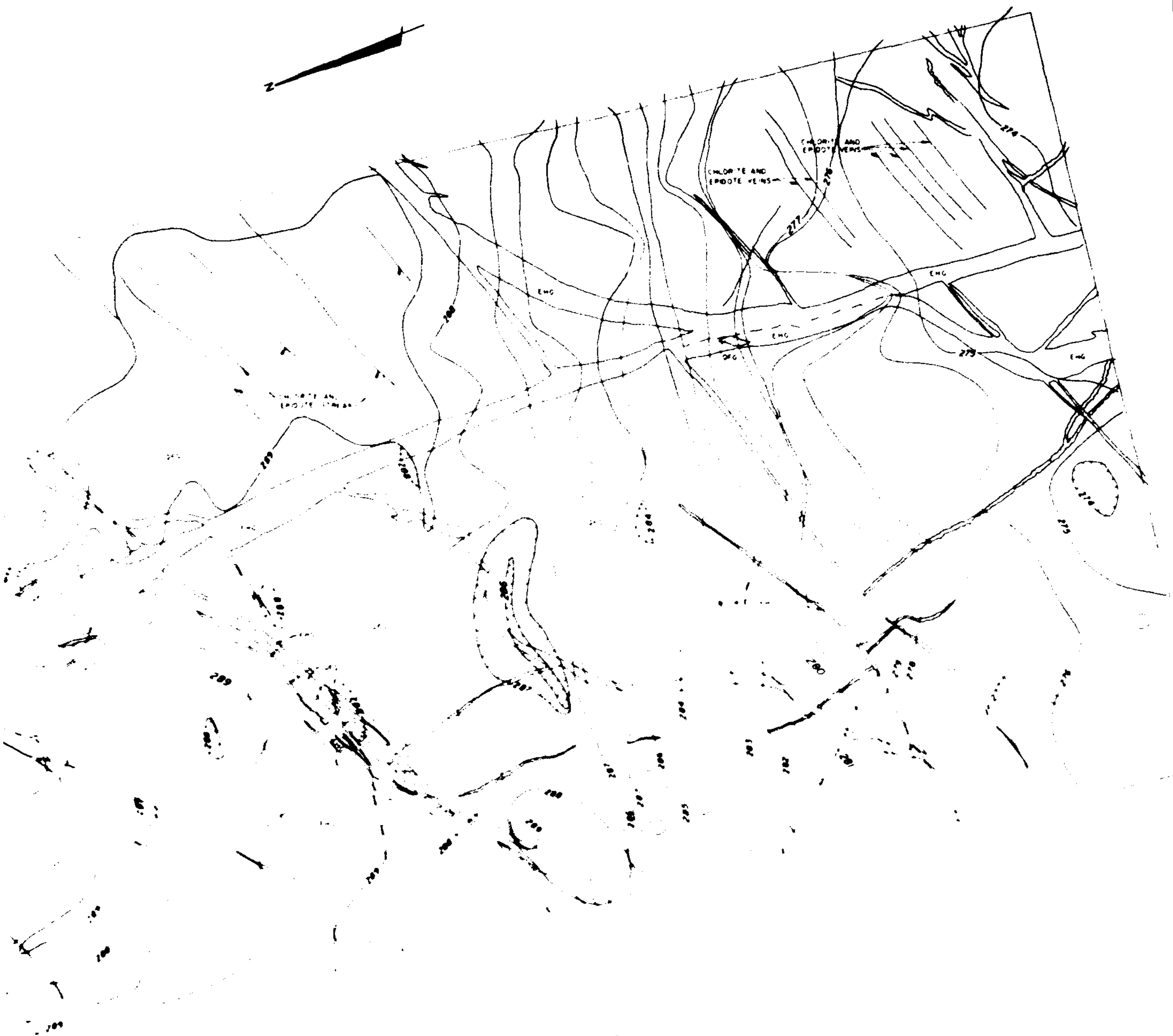
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 8
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 26

REVISIONS



5
4
D
C
B
A
04+76
04+66
04+6
04+56
04+46
04+41
04+36
282
283
284
285
286
287
288
289
MONO



NOTE For legend see Monolith 1

MONOLITH 9

280 Approximate top of foundation
Contour interval - 1.0 foot (elevation MSL)

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CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT

CONCRETE DAM
MONOLITH 9
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

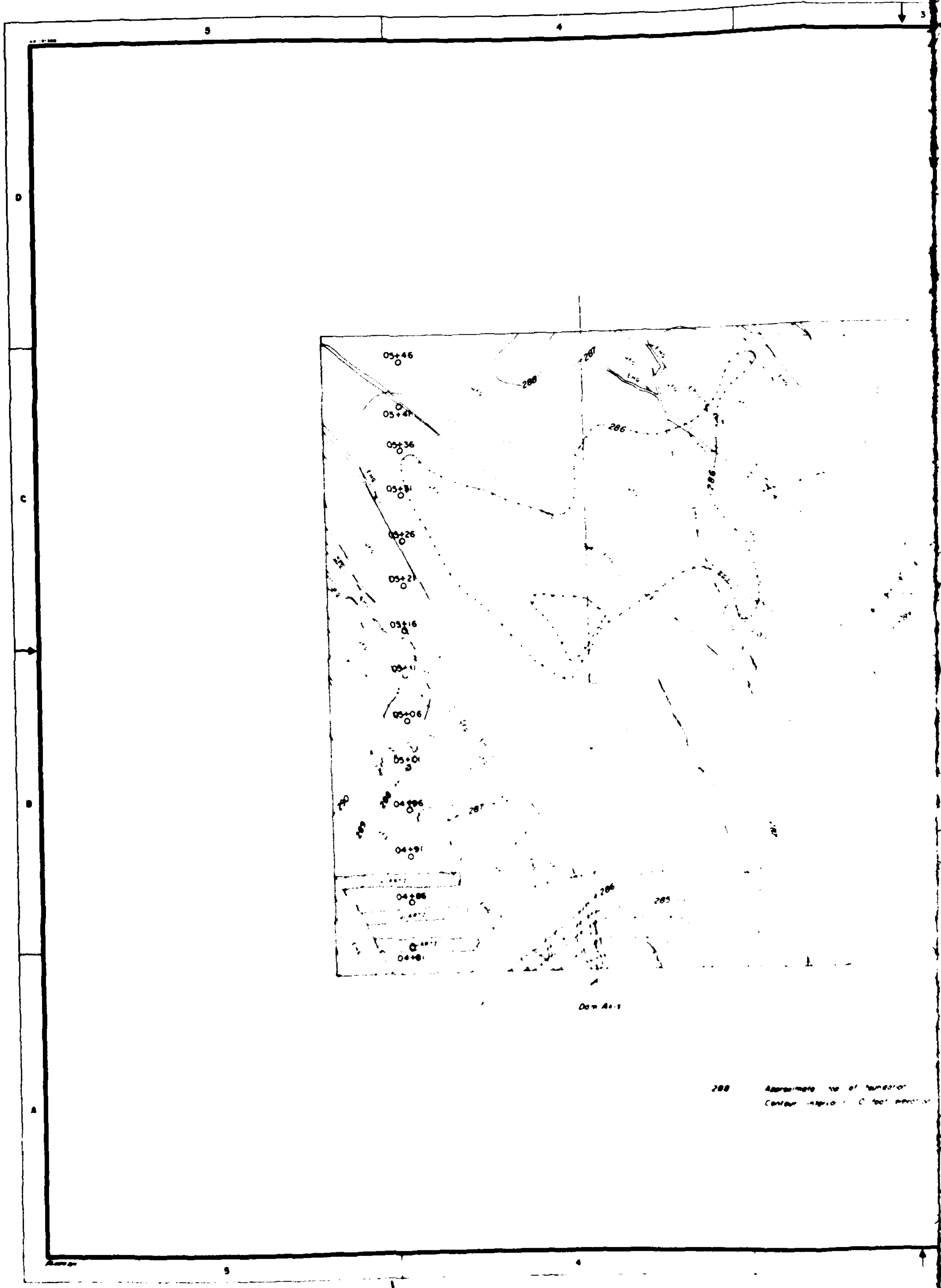
PLATE 27

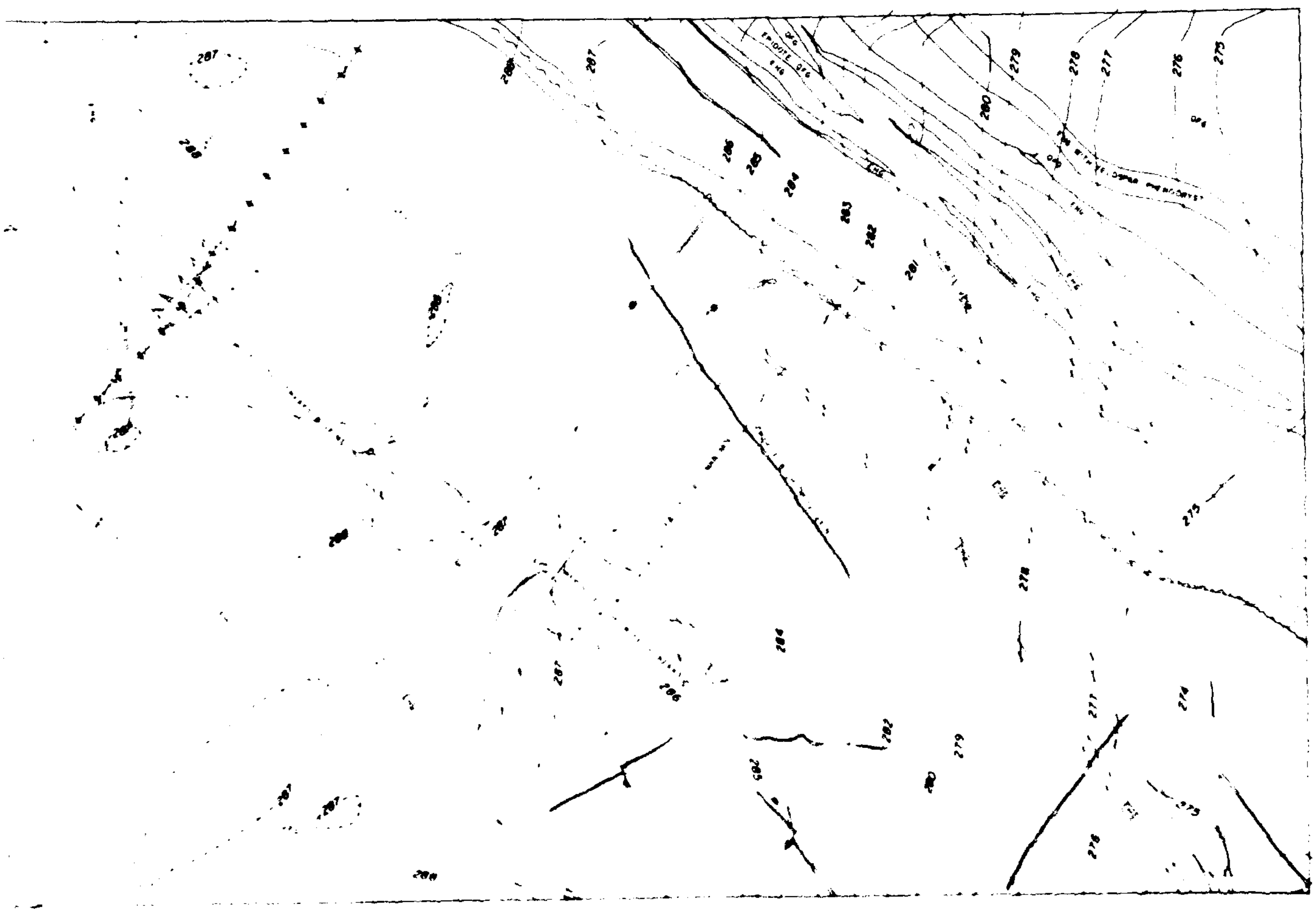
SCALE AS SHOWN

REVISIONS

27

2


05+46
05+41
05+36
05+31
05+26
05+21
05+16
05+11
05+06
05+01
04+96
04+91
04+86
04+81
288
287
286
285
Dam Axis



MONOLITH 10

SCALE 1" = 5'

NOTE: For legend see Monolith 1

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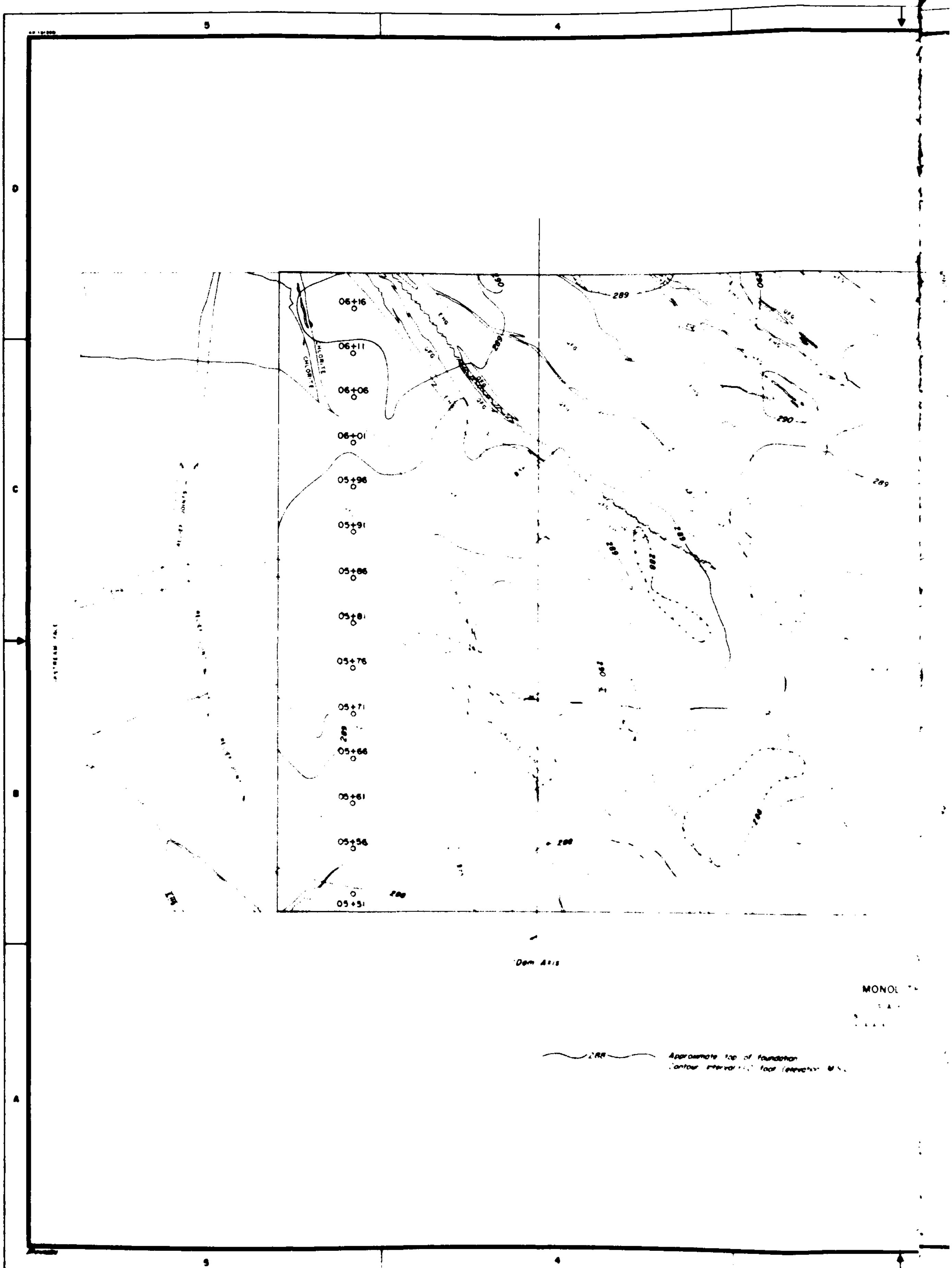
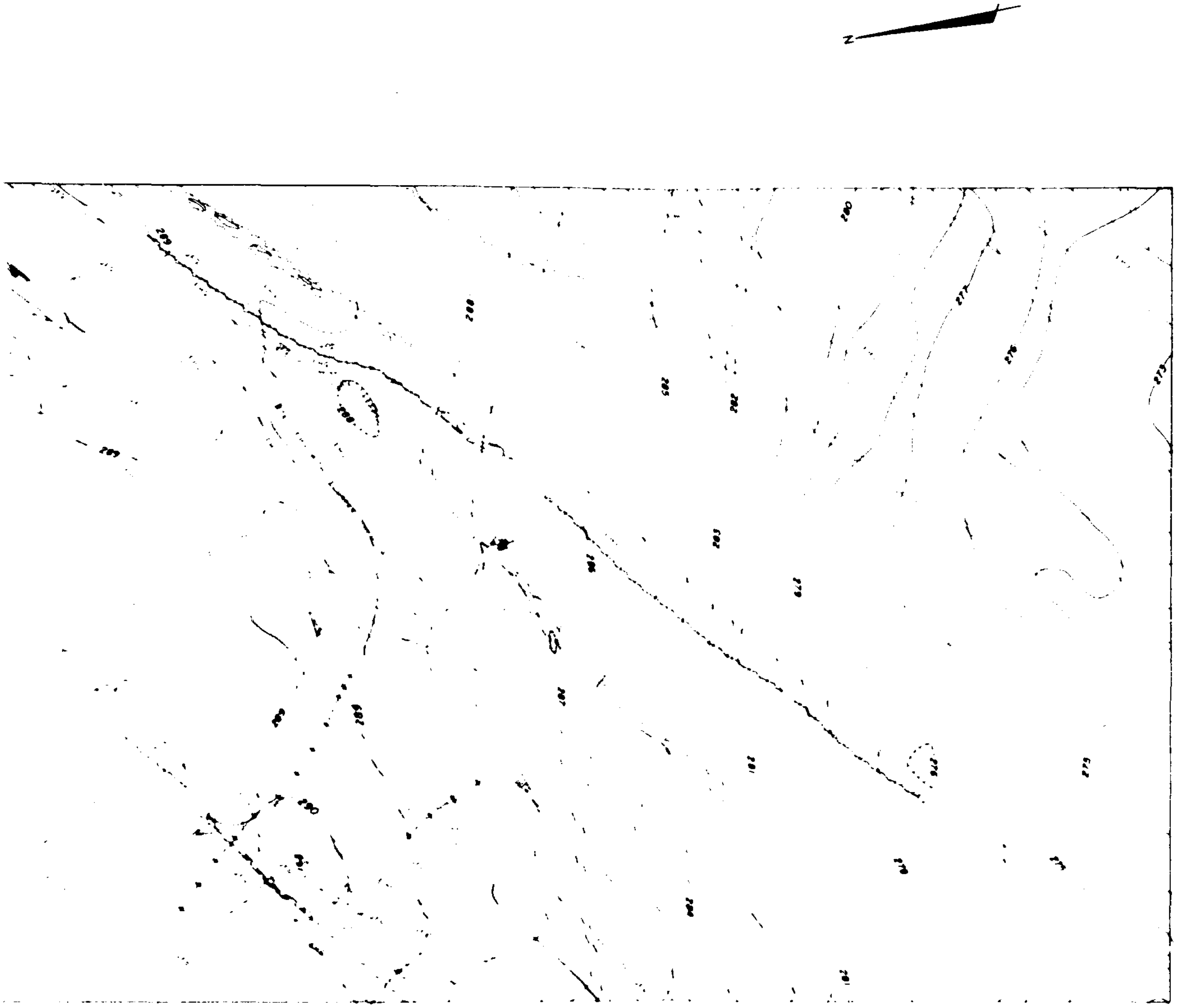
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 10
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER — GEORGIA AND SOUTH CAROLINA

C			28
SCALE AS SHOWN		SHEET	

28


5
4
D
C
B
A
06+16
06+11
06+06
06+01
05+96
05+91
05+86
05+81
05+76
05+71
05+66
05+61
05+56
05+51
CHLORITE
289
290
288
Dam Axis
Approximate top of foundation



MONOLITH 11

NOTE
For legend see Monolith

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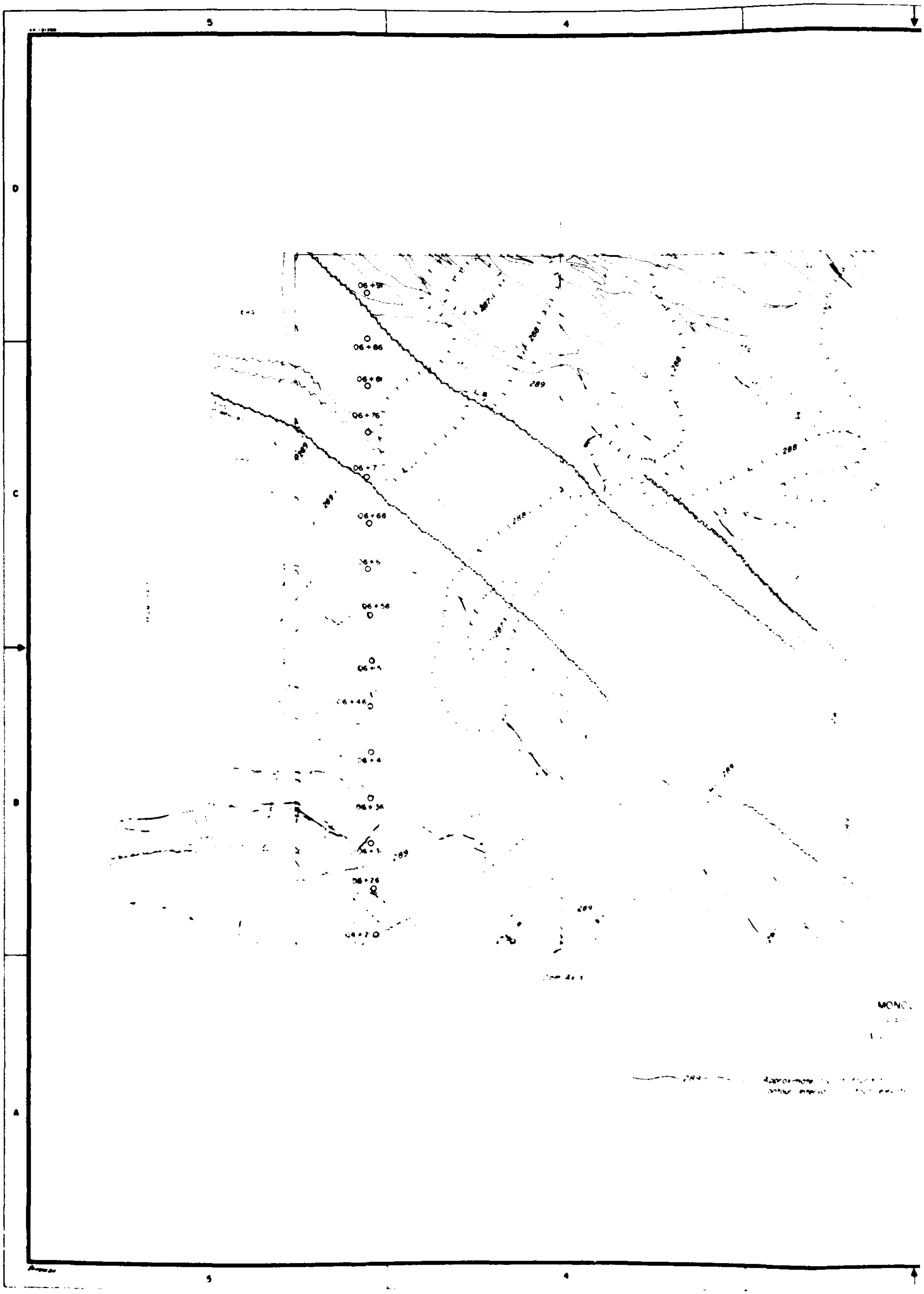
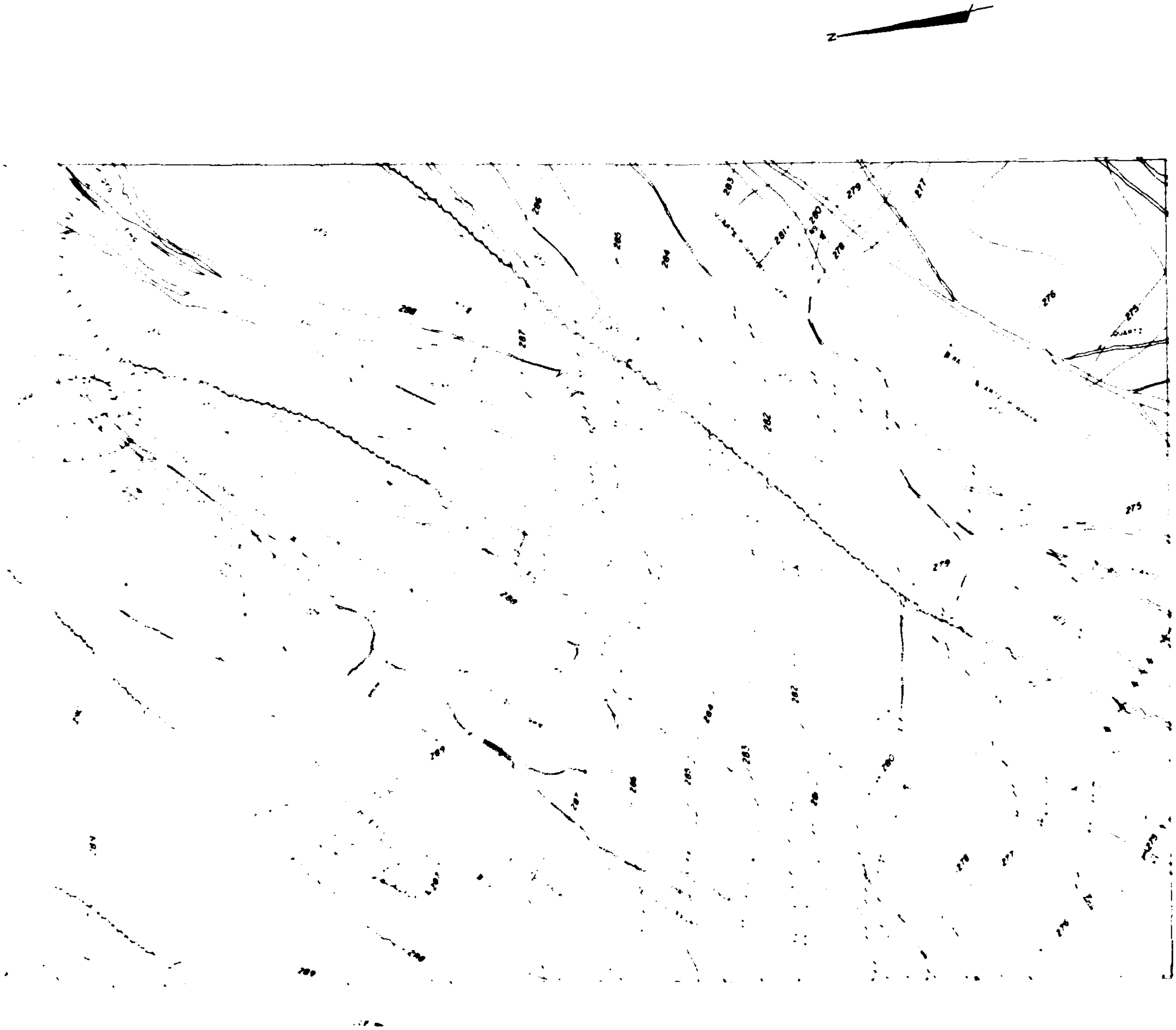
FOUNDATION REPORT

CONCRETE DAM
MONOLITH 11
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 29



5
4
D
C
B
A
06+86
06+81
06+76
06+7
06+66
06+56
06+4
06+36
06+26
287
288
289



MONOLITH 12

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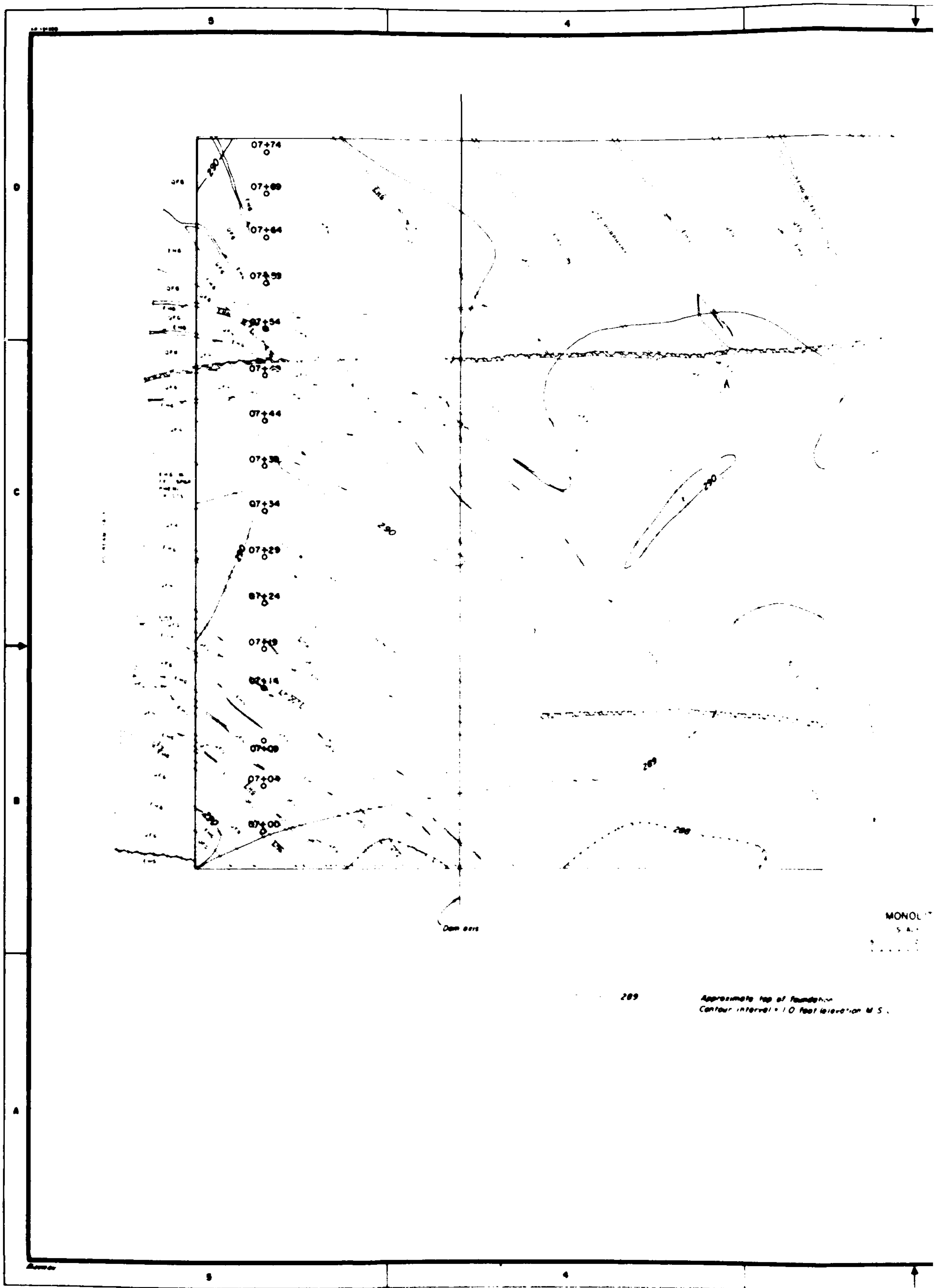
CONCRETE DAM
MONOLITH 12
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

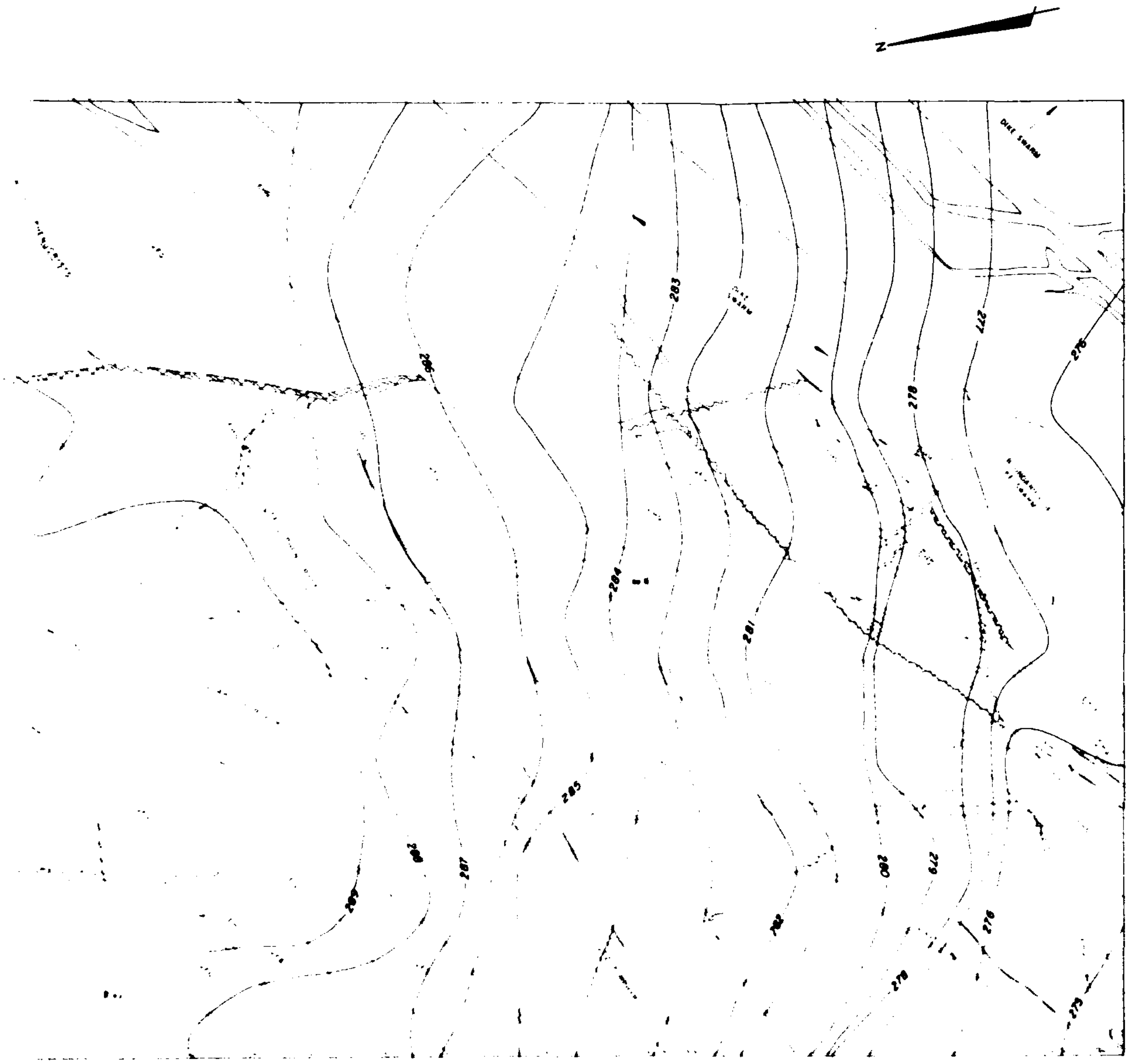
PLATE 30

(30)

2



07+74
07+69
07+64
07+59
07+54
07+44
07+34
07+29
07+24
07+04
07+00
290
289
288
Dam axis
MONOL
289
Approximate top of foundation
Contour interval = 1.0 foot (elevation M.S.



MONOLITH 13
SCALE 1"=5'

NOTE
For legend see Monolith 1

Top of foundation
... foot elevation M.S.L.

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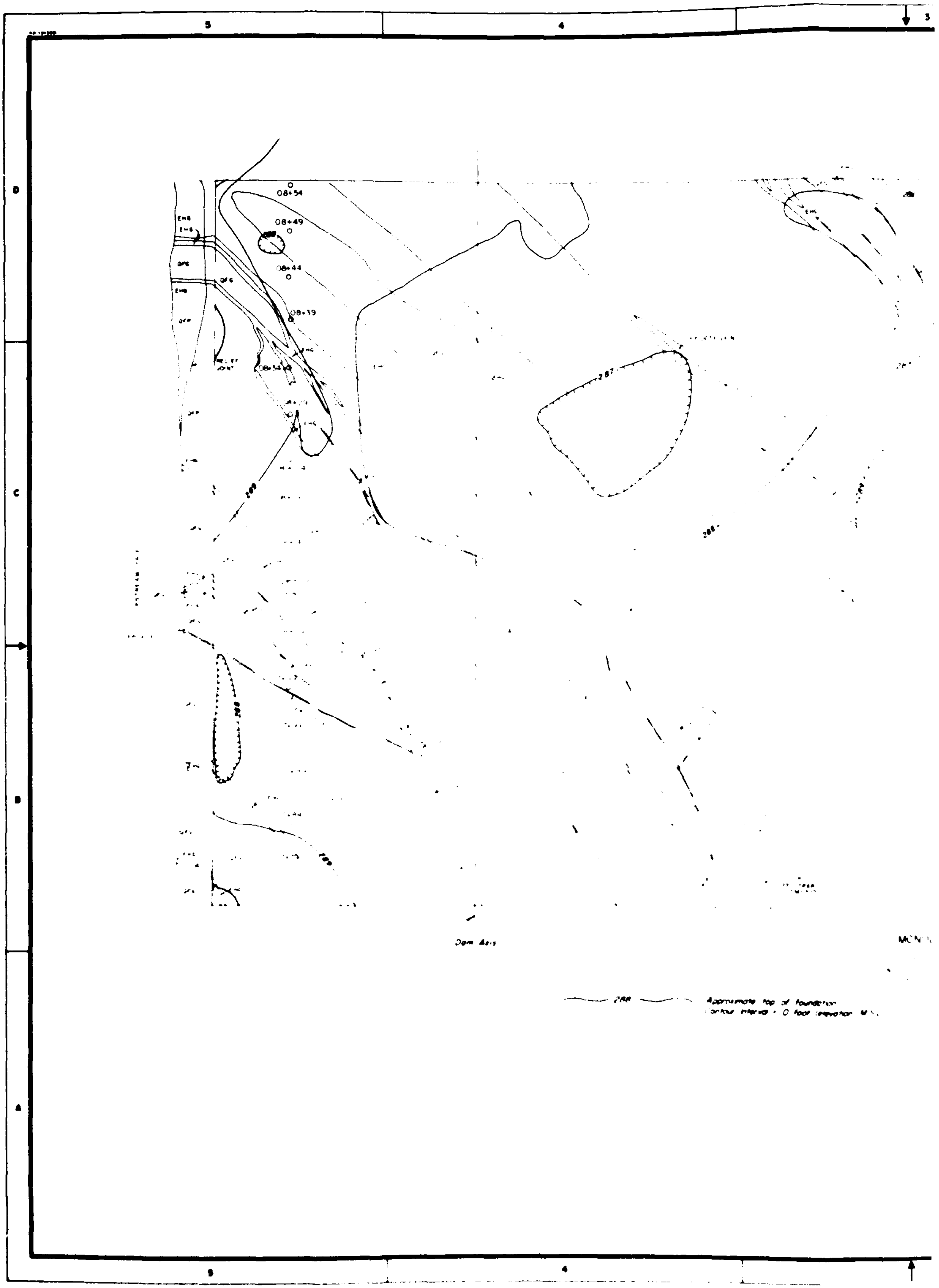
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 13
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE F
PLATE 34
SCALE AS SHOWN

REVISIONS



5
4
3
D
C
B
A
08+54
08+49
08+44
08+39
EHG
QFG
RELIEF JOINT
287
288
Dam Axis
288
Approximate top of foundation

3 | 2 | 1

MONOLITH 14

SCALE 1:5

... foundation
... elevation M.S.L.

NOTE
For legend see Monolith ...

REVISIONS	

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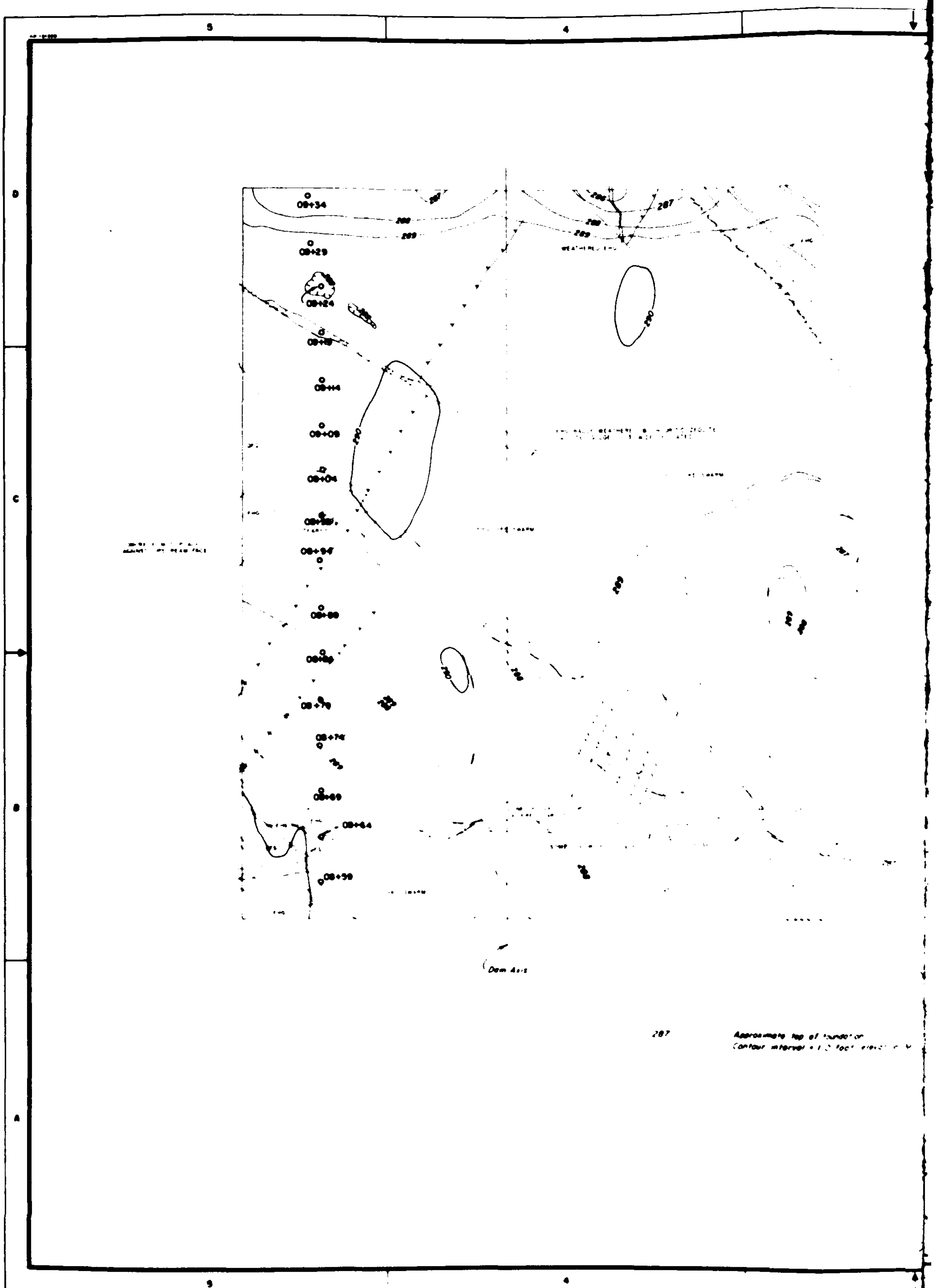
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 14
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

F | PLATE 32

3 | 2 | 1


5
4
D
C
B
A
08+34
08+29
08+24
08+14
08+09
08+04
08+89
08+69
08+64
08+59
290
Dam Axis
287
Approximate top of foundation

NOTE
For legend see Monolith 1

...mate top of foundation
...interval 1.0 foot (elevation MSL)

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FOUNDATION REPORT
CONCRETE DAM
MONOLITH 15
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

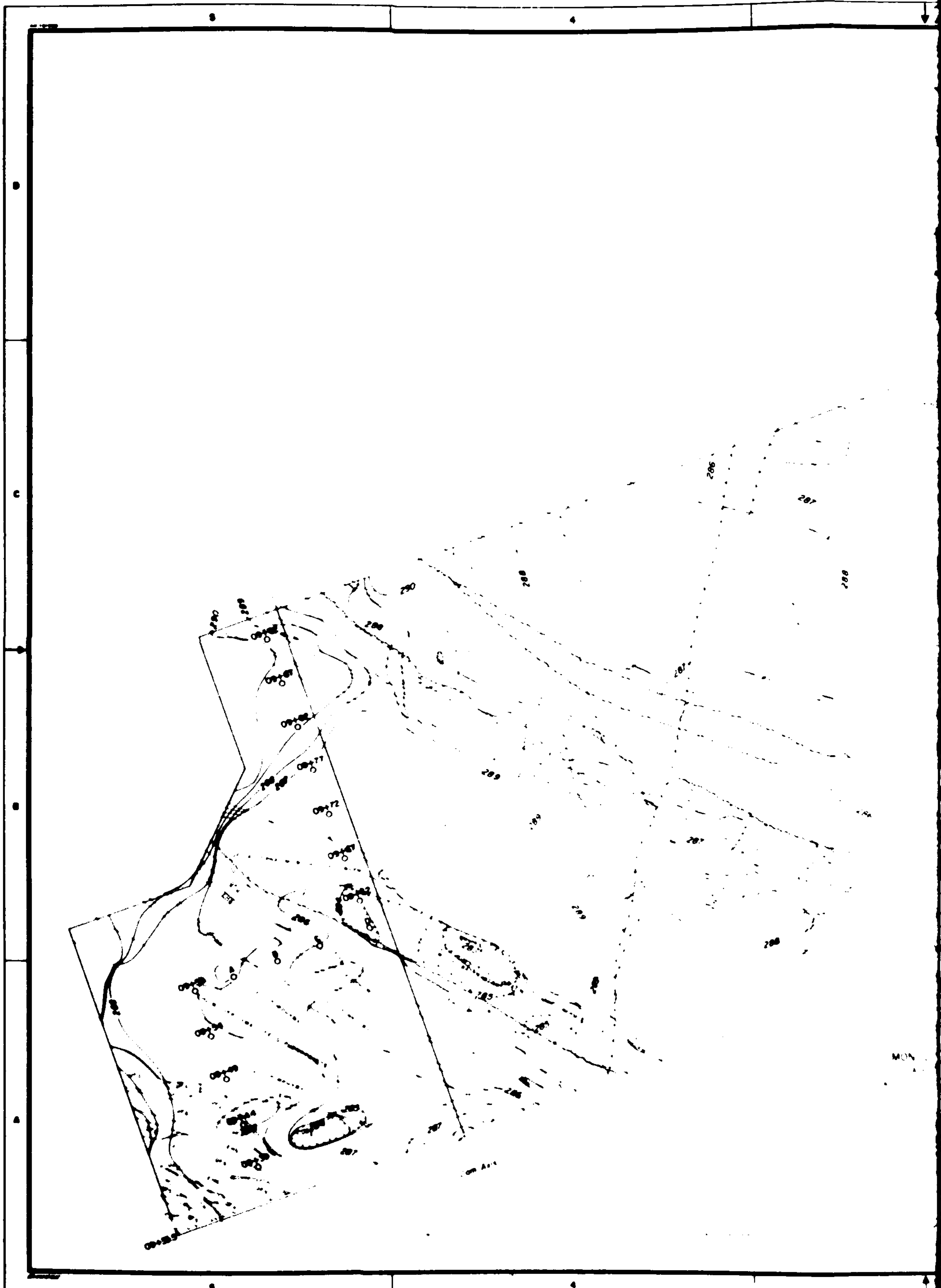
PLATE 33

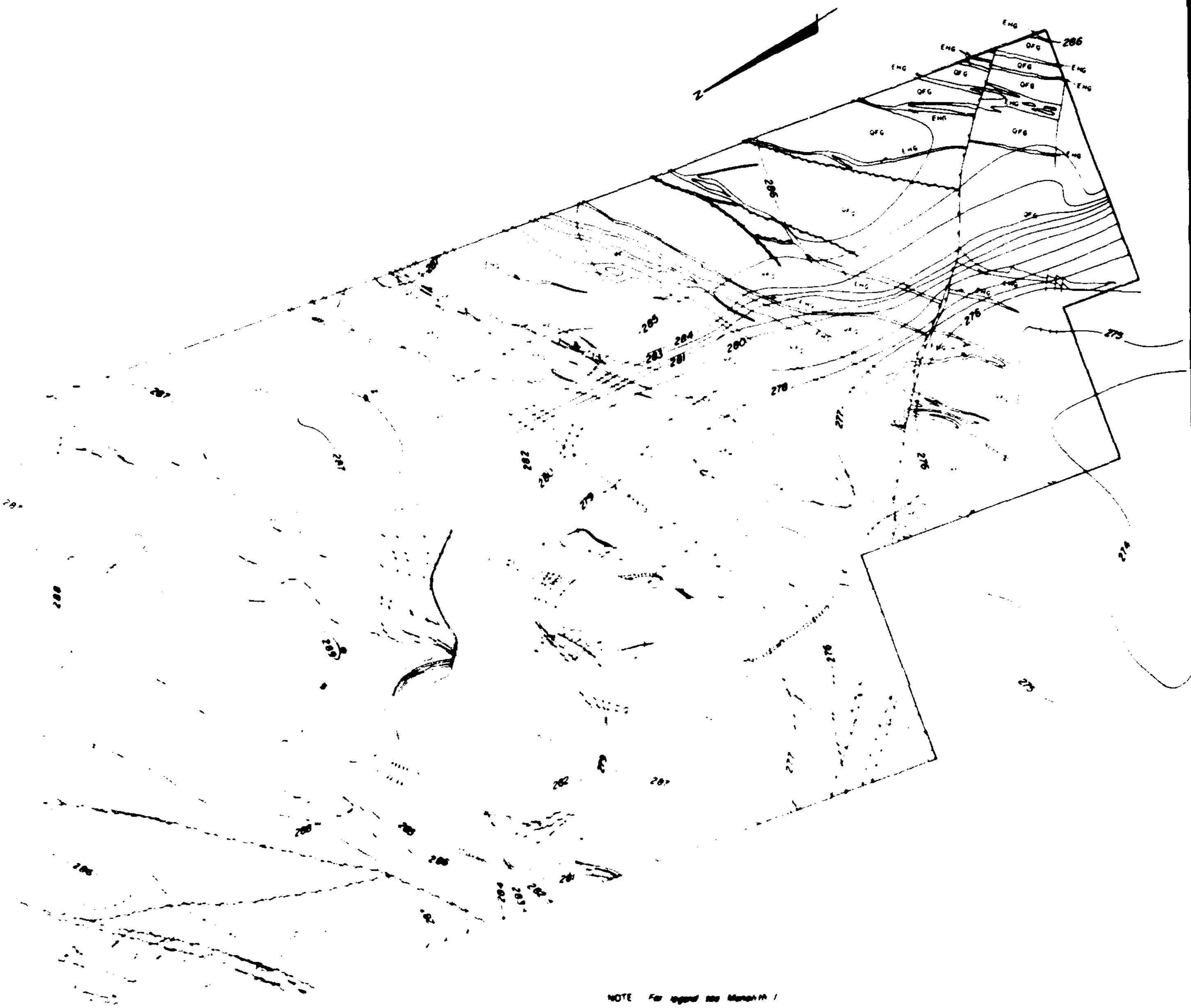
SCALE AS SHOWN

REVISIONS

33

2





NOTE For legend see Monolith 1

284 Approximate top of foundation
Contour interval = 1.0 foot (elevation M.S.L.)

MONOLITH 16
SCALE 1:5

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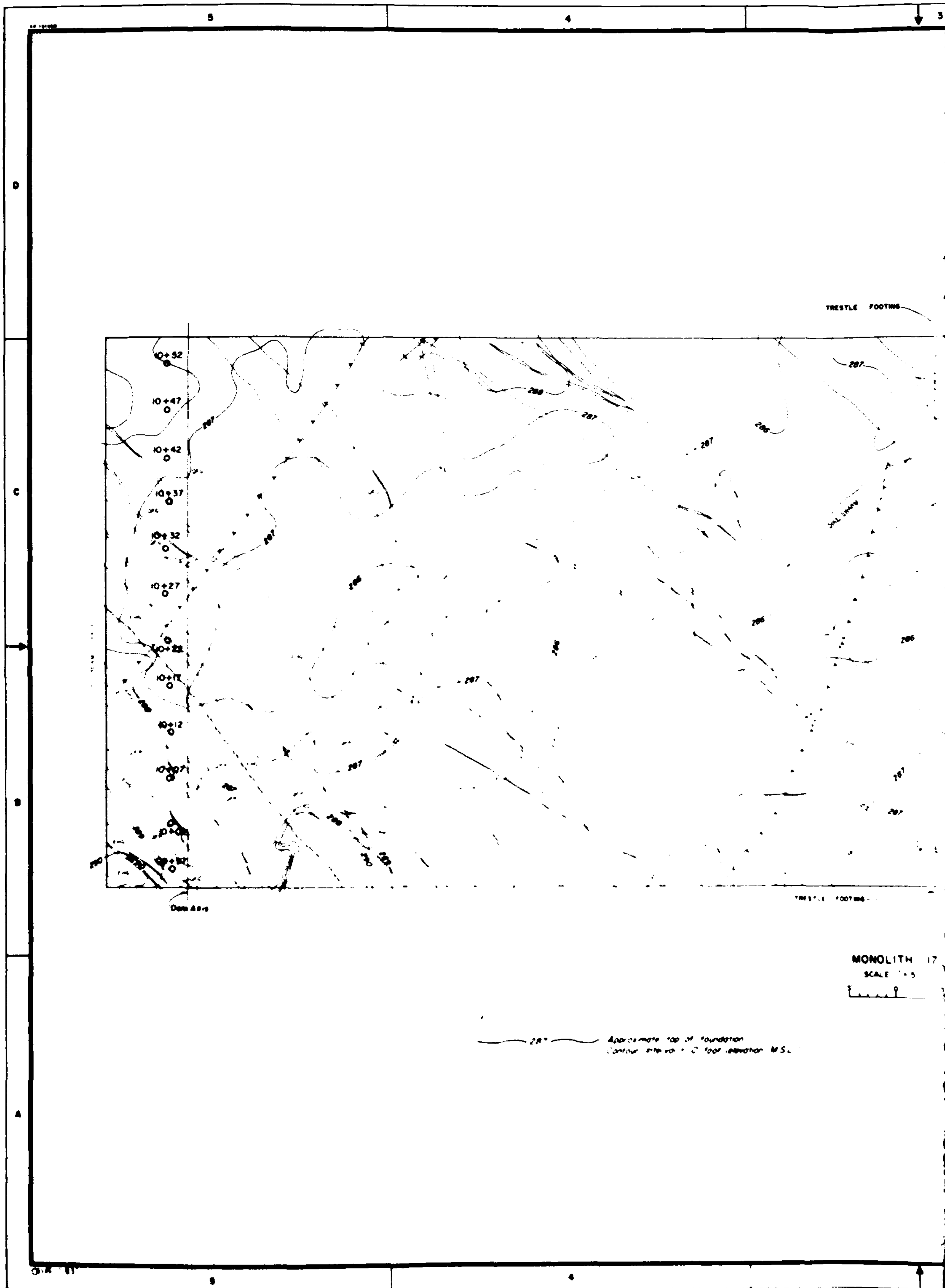
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 16
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE			PLATE
F			34

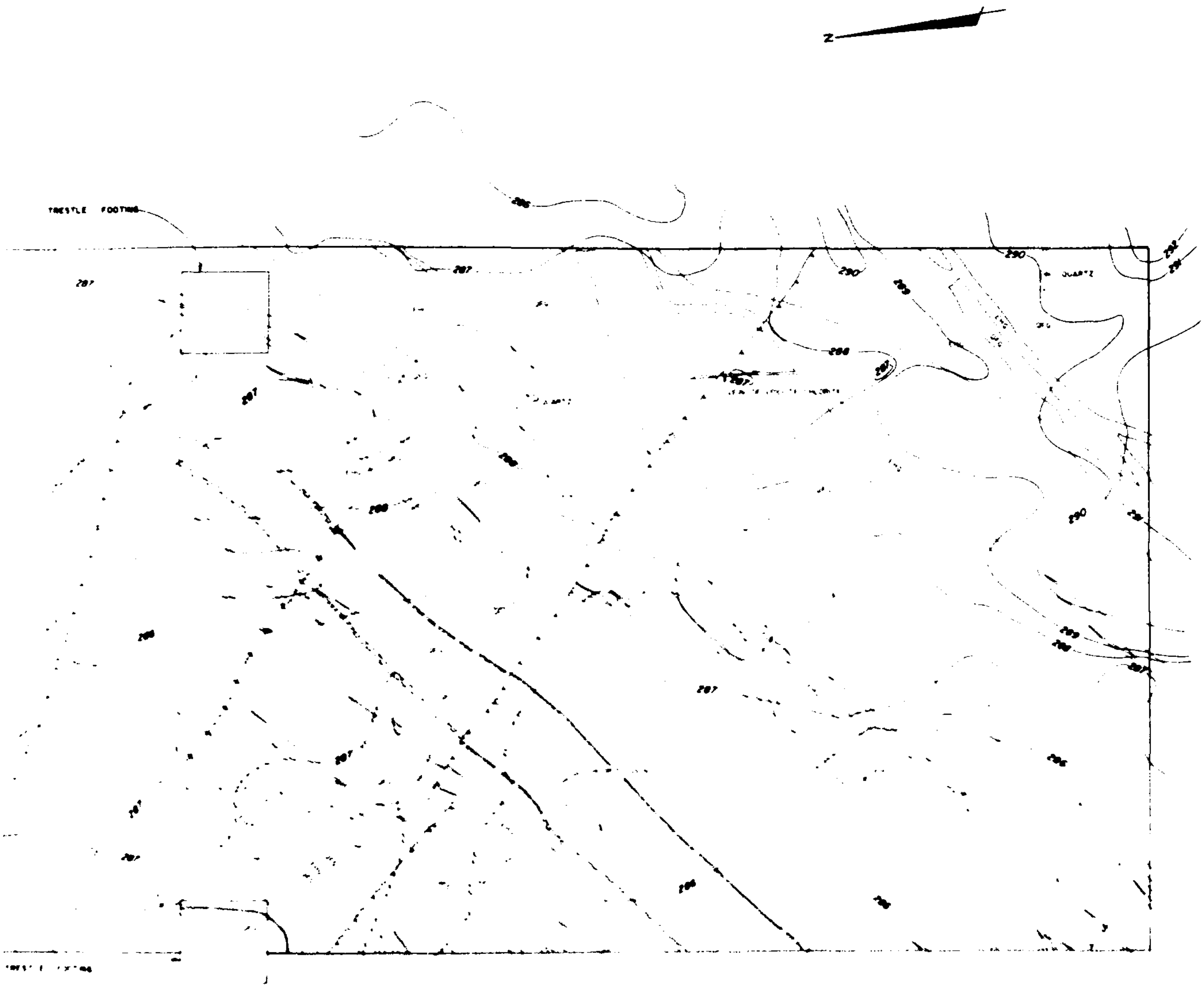
SCALE AS SHOWN

REVISIONS

34



TRESTLE FOOTING
10+52
10+47
10+42
10+37
10+32
10+27
10+22
10+17
10+12
10+07
10+02
287
288
286
Dam Axis
TRESTLE FOOTING
MONOLITH 17
287 Approximate top of foundation
Contour interval 1.0 foot elevation M.S.L.



MONOLITH 17

SCALE 1:5

NOTE
For legend see Monolith

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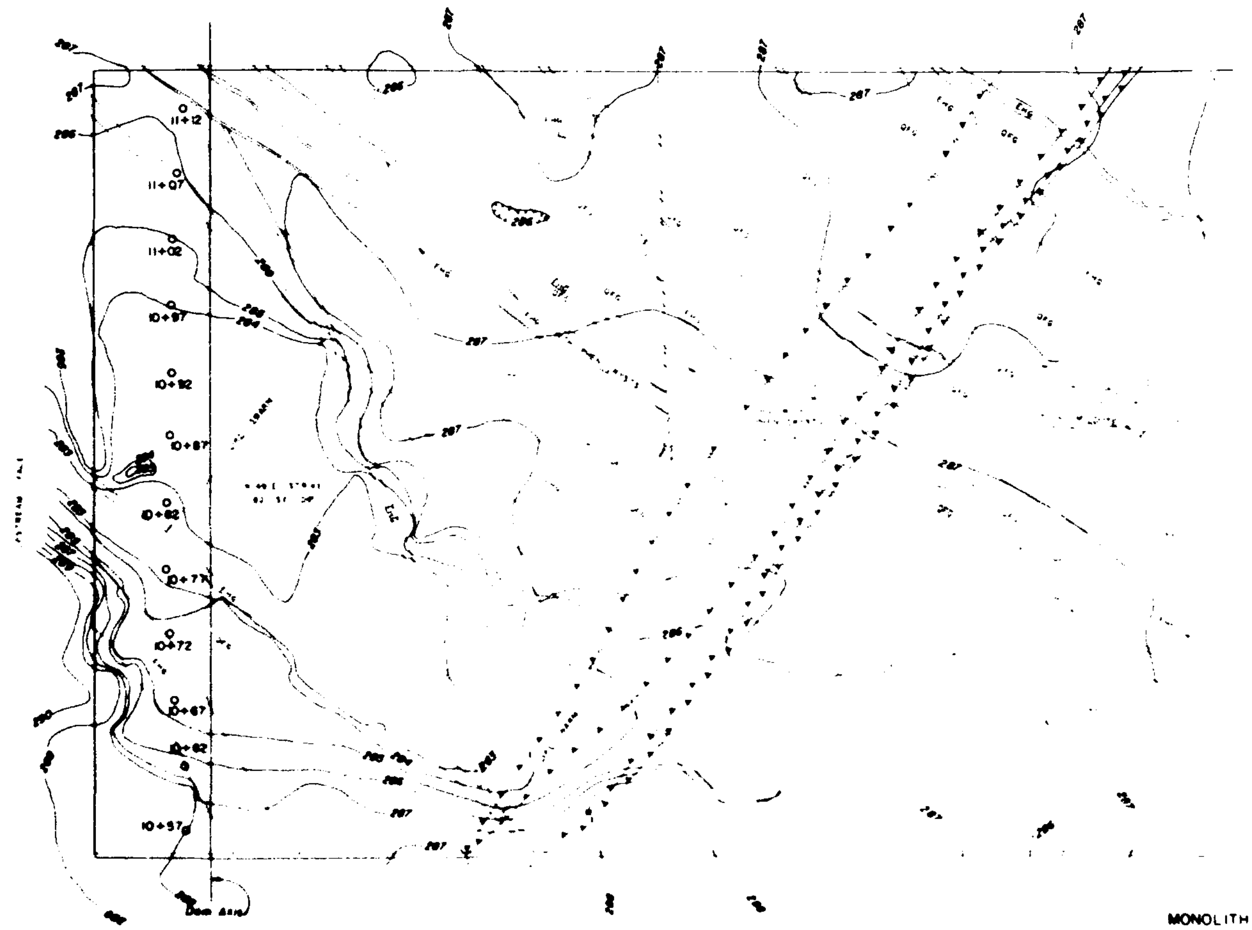
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 17
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 35

REVISIONS

2

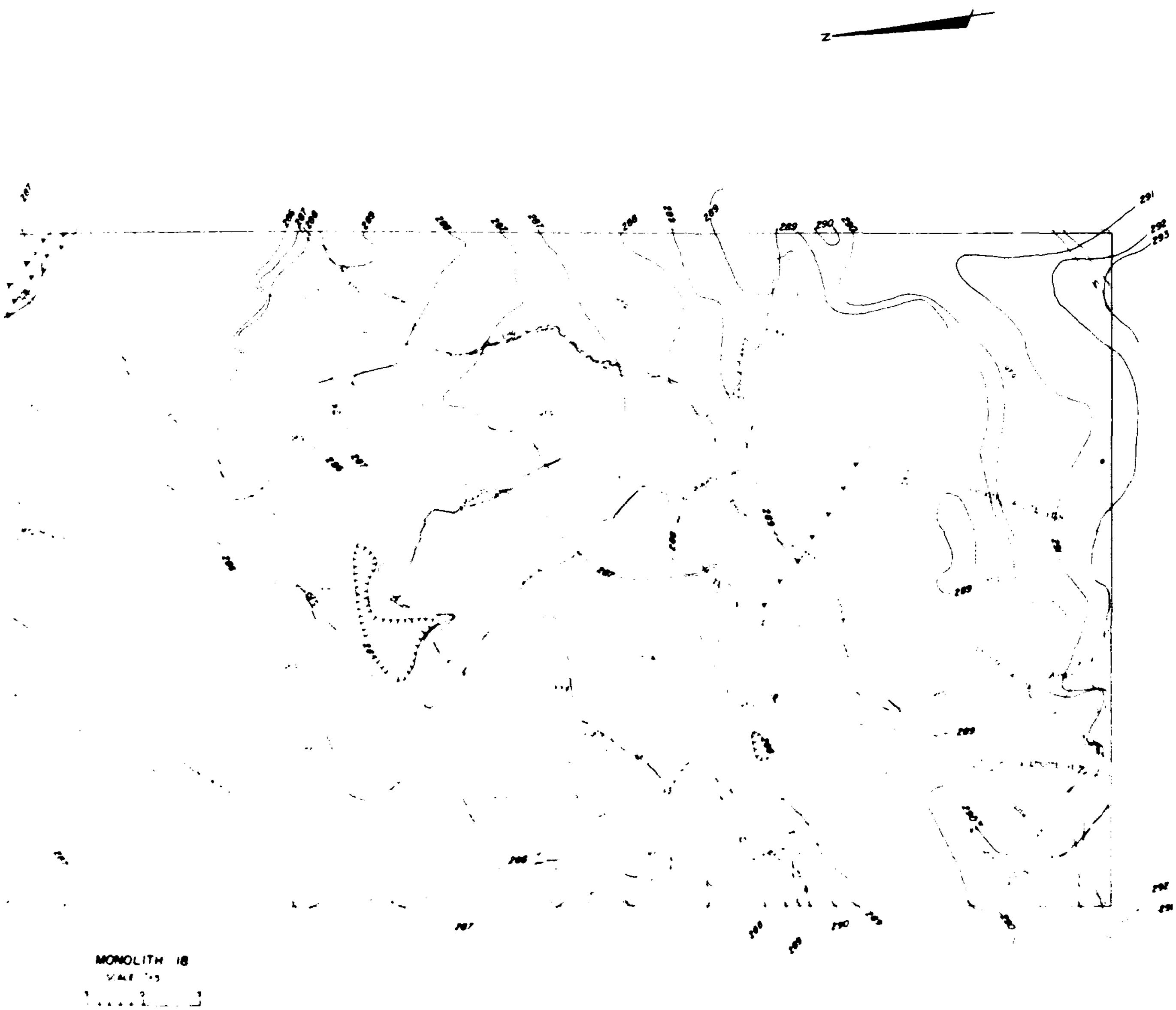


MONOLITH

SCALE

288 Approximate top of foundation

Contour interval 1.0 foot (elevation MSL)



NOTE: For legend see Monolith 1

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FOUNDATION REPORT

CONCRETE DAM
MONOLITH 18

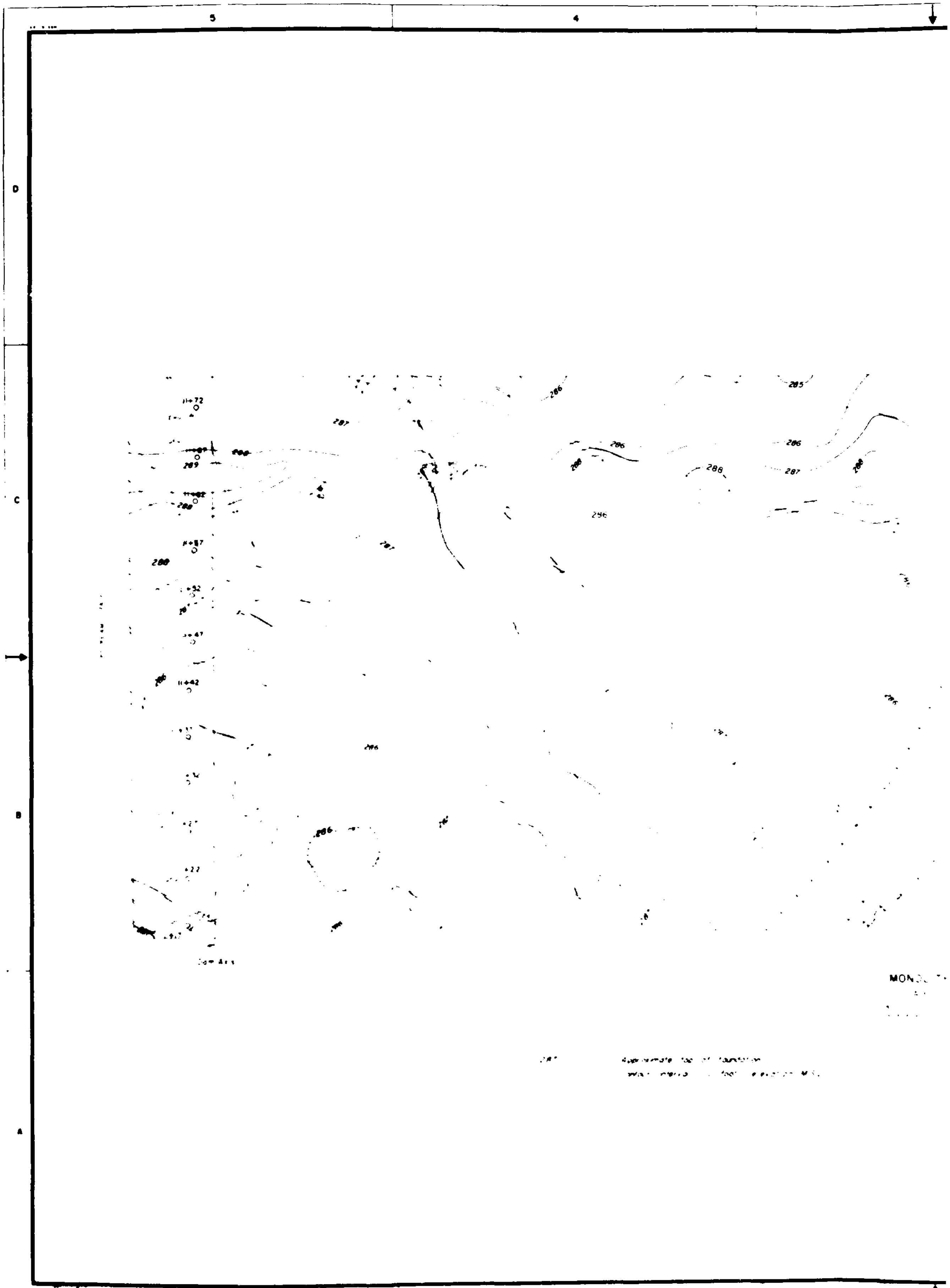
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 36

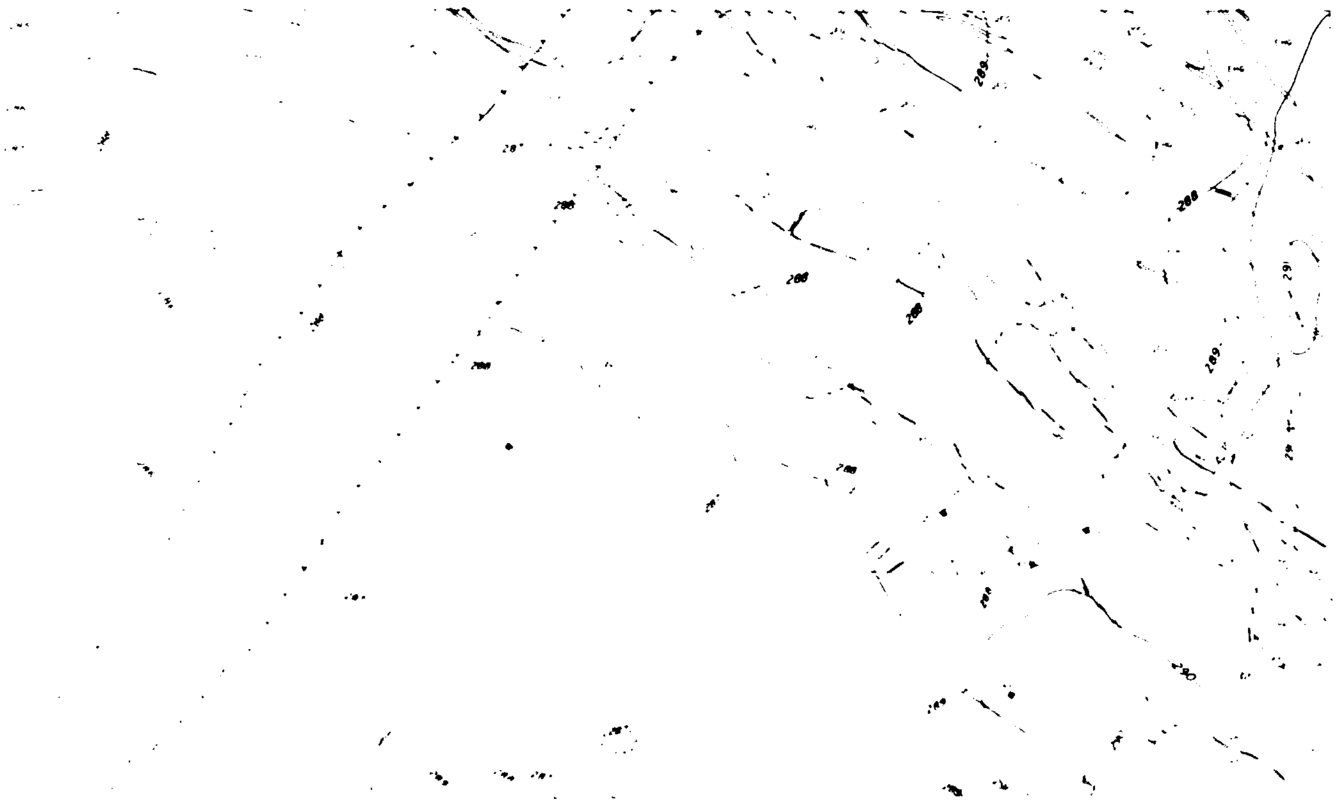
SCALE AS SHOWN

REVISIONS

2



5
4
D
C
B
A
285
286
287
288
289
11+72
11+62
11+57
11+42

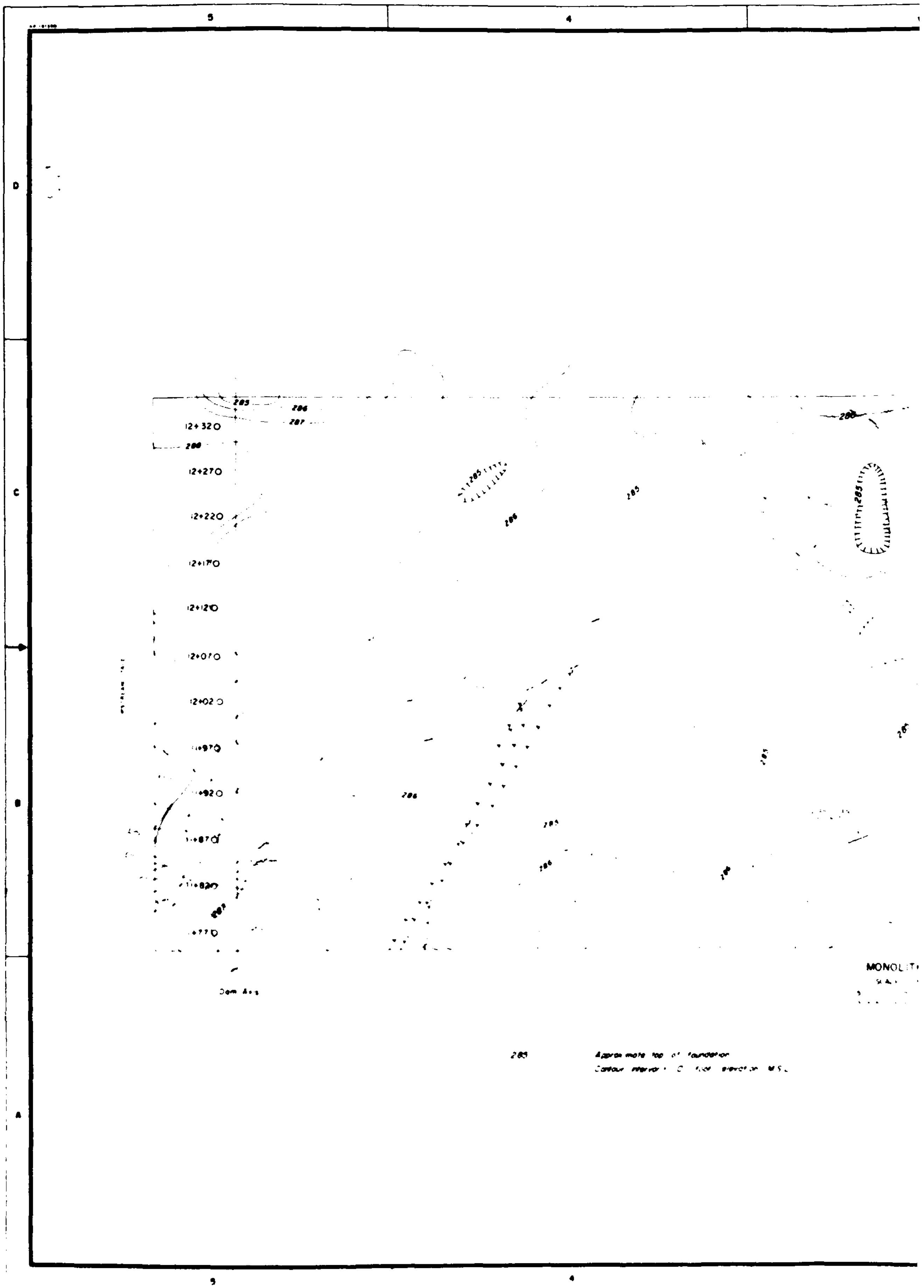


FOUNDATION REPORT

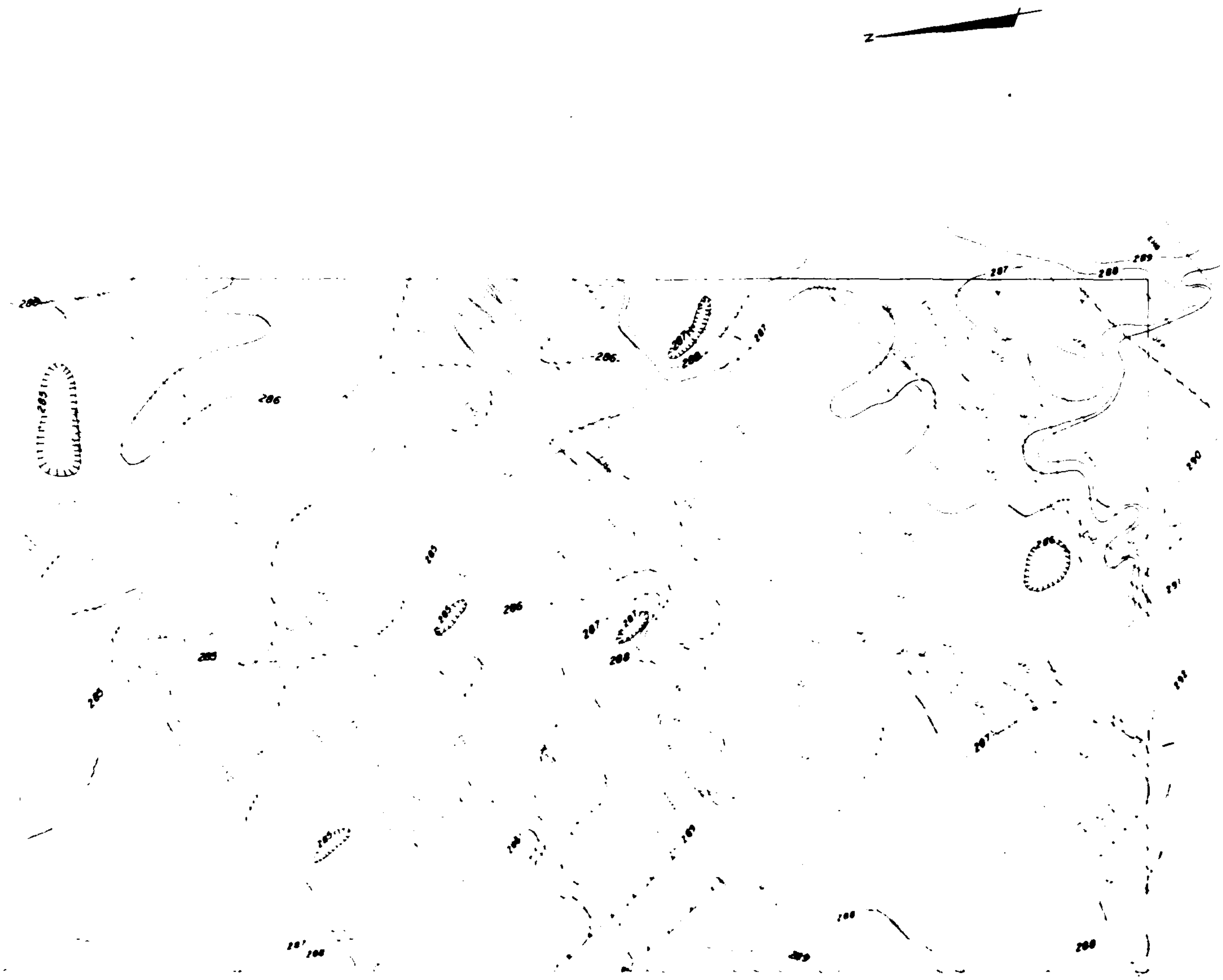
CONCRETE DAM

MONOLITH 19

SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA



12+32
12+27
12+22
12+17
12+12
12+07
12+02
Dam Axis
285
286
287
288
MONOLITH
285 Approximate top of foundation



MONOLITH 20

NOTE: For legend see Monolith

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FOUNDATION REPORT

CONCRETE DAM

MONOLITH 20

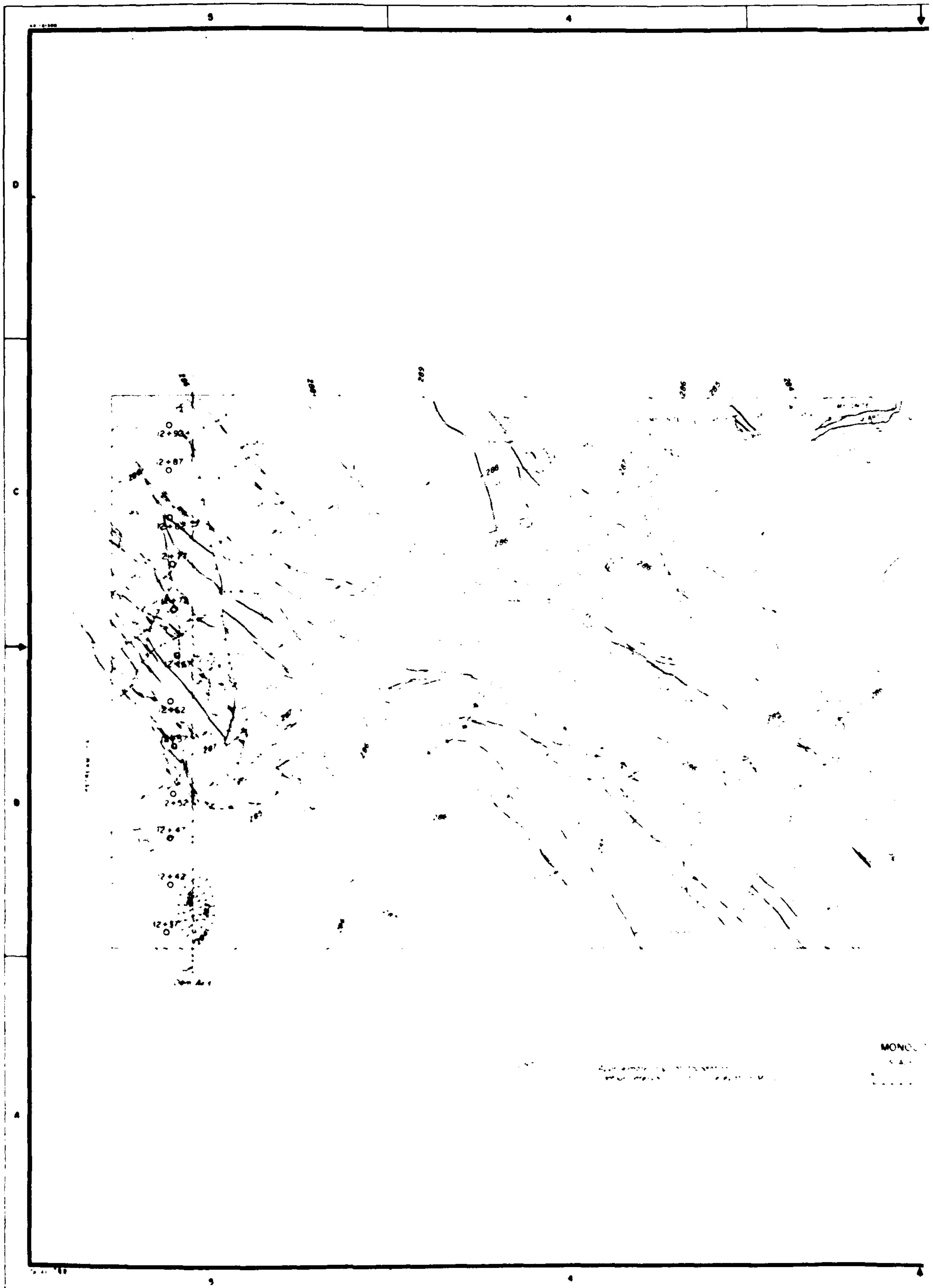
RICHARD B. RUSSELL PROJECT

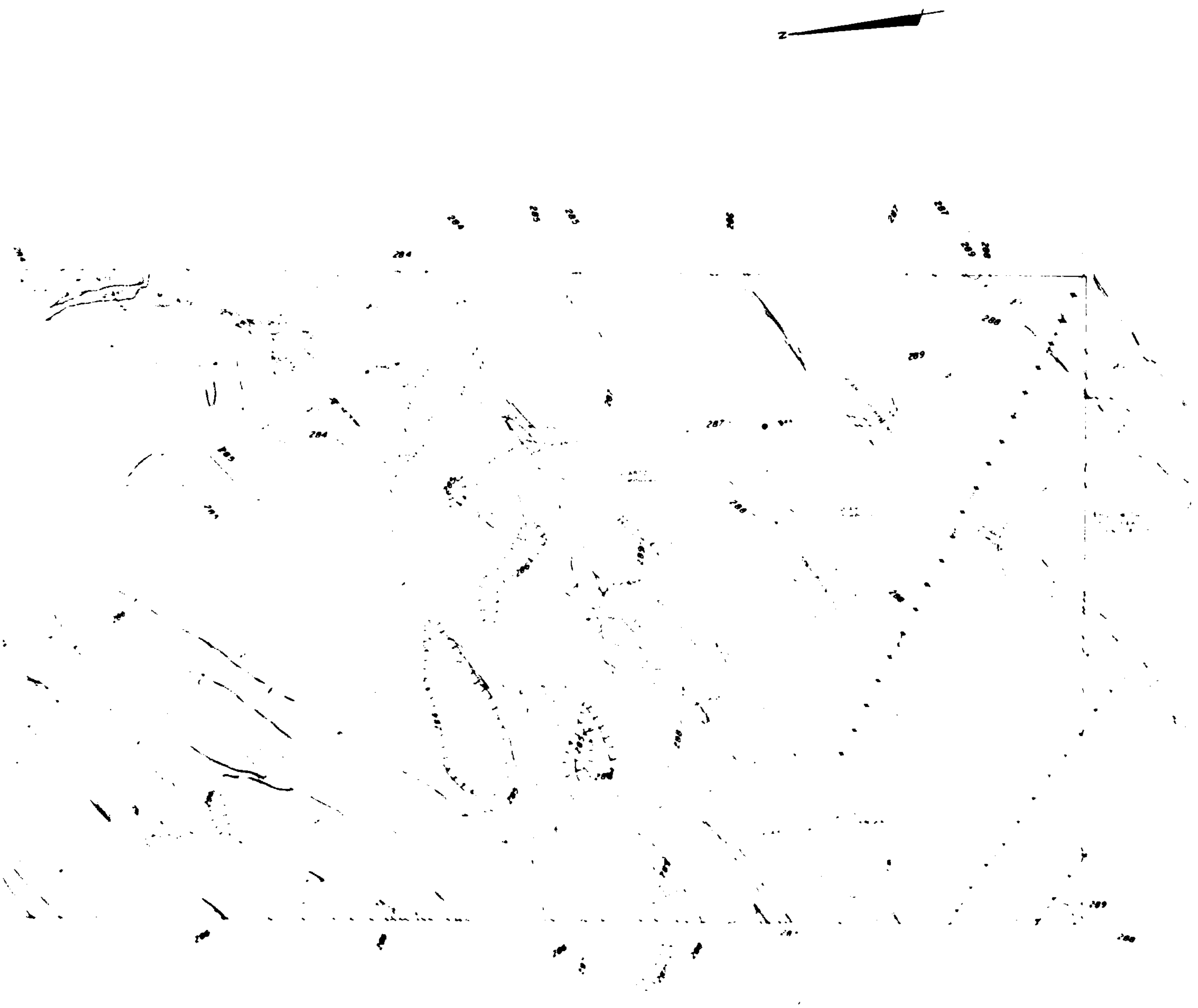
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 38

SCALE AS SHOWN

2





MONOLITH 21

NOTE
For legend see Monolith

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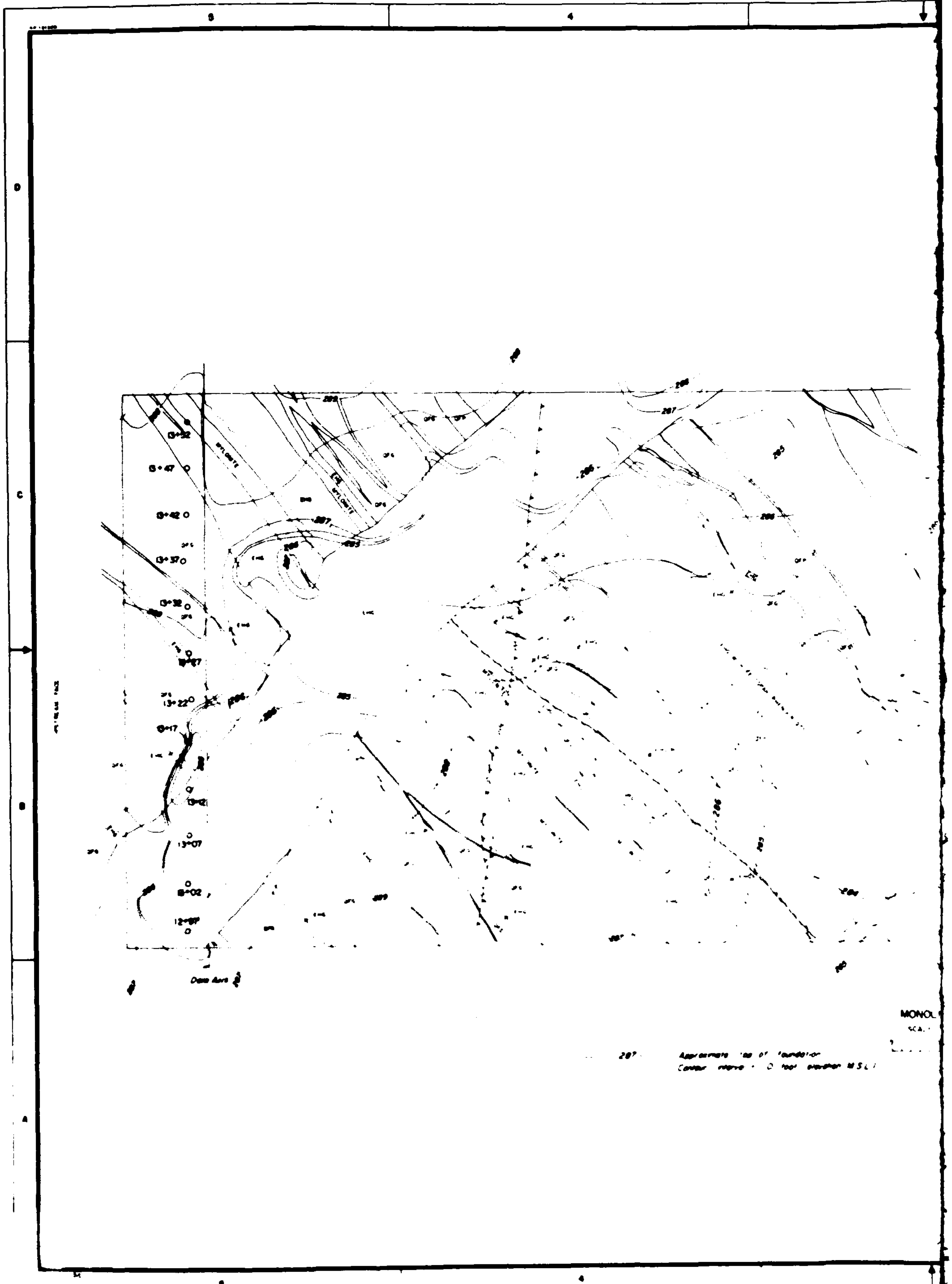
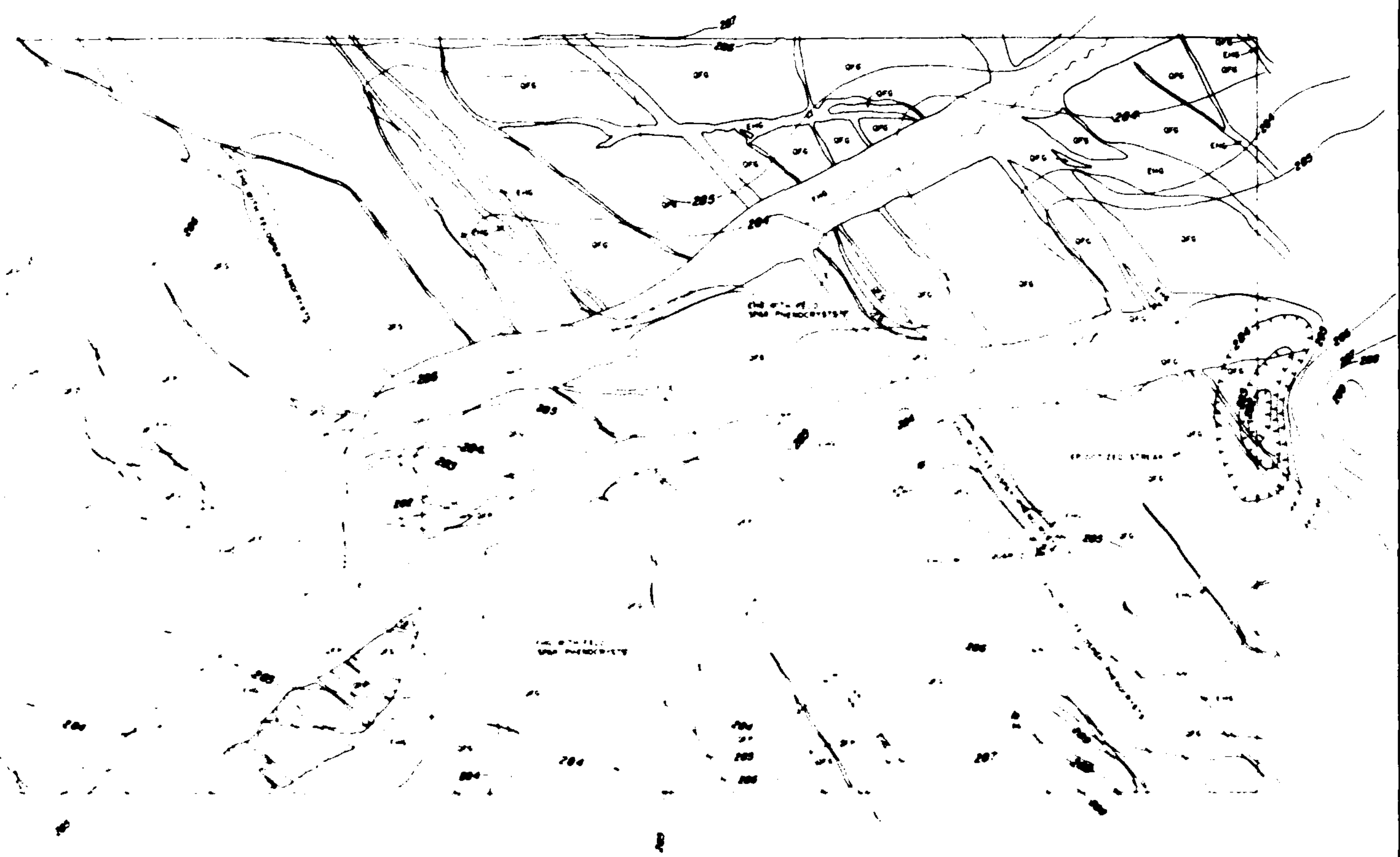
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 21
RICHARD B. RUSSELL PROJECT

SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 39



13+52
13+47
13+42
13+37
13+32
13+22
13+17
13+07
13+02
12+97
Dam Axis
MONOL



MONOLITH 22

SCALE 1:5

NOTE For legend see Monolith 1

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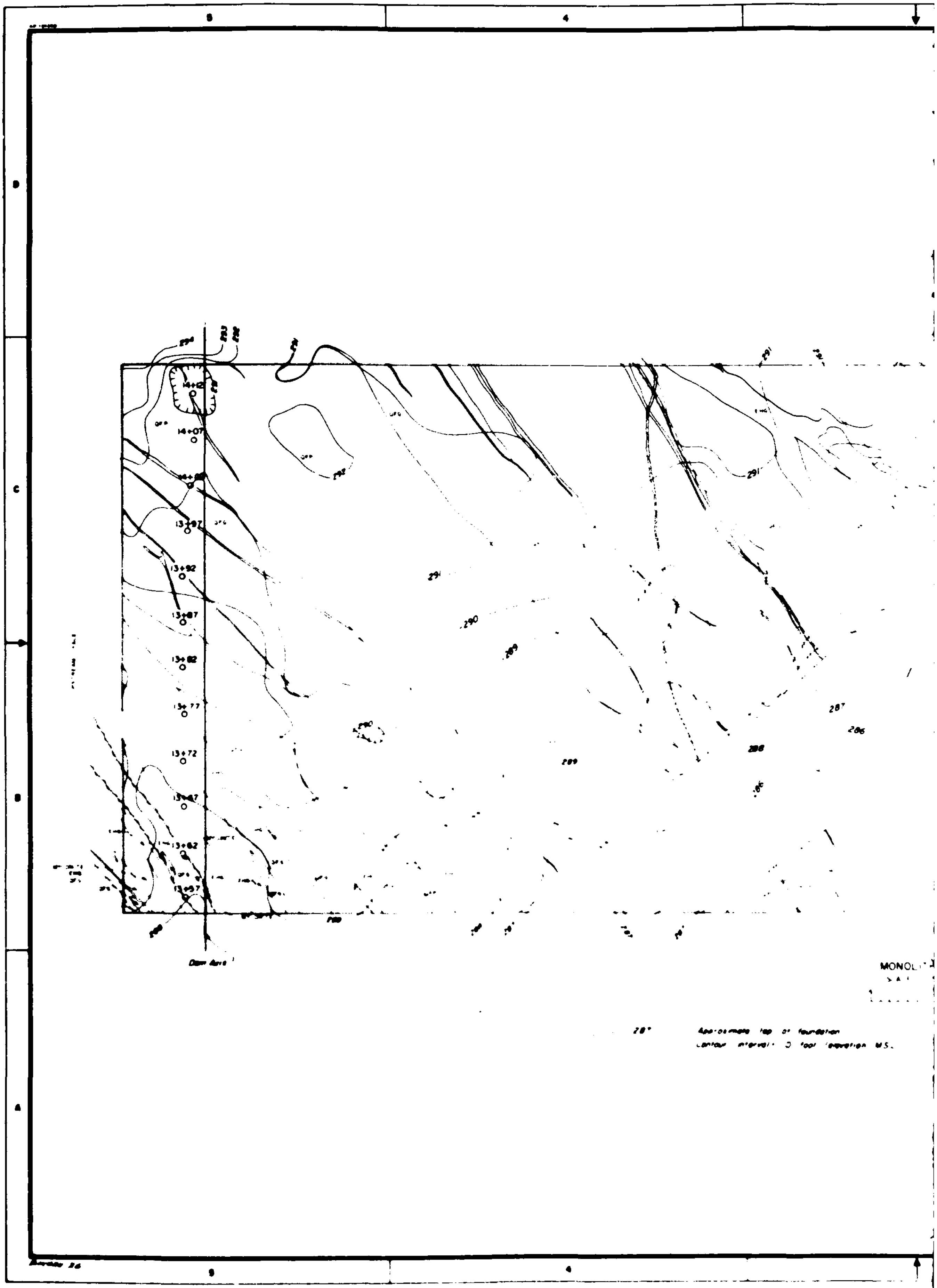
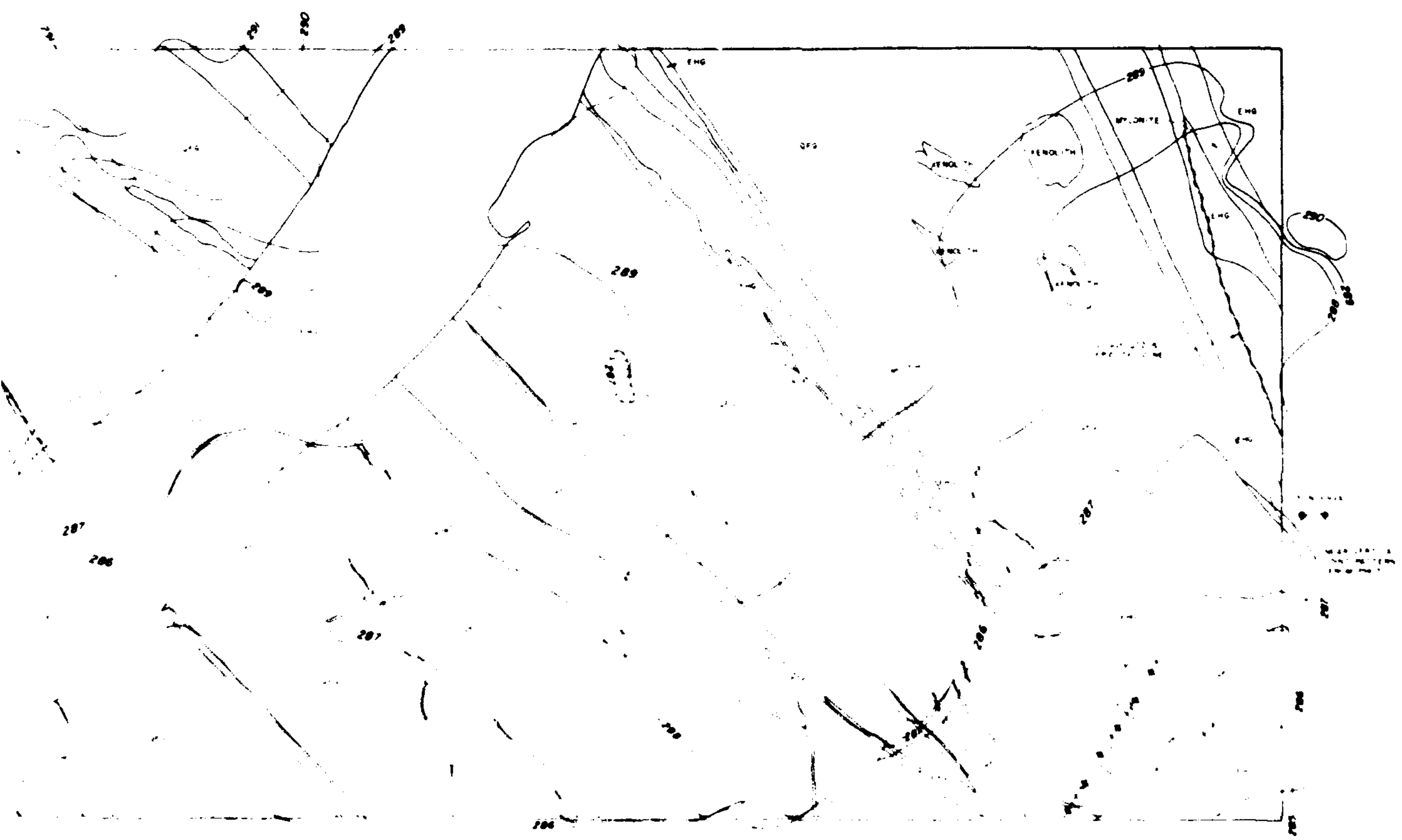
CONCRETE DAM
MONOLITH 22
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 40

REVISIONS

40


14+12
14+07
13+97
13+92
13+87
13+82
13+77
13+72
13+67
13+62
13+57
292
291
290
289
288
287
286
Dam Axis
MONOLI
5
4
B
C
D
A
28*
Approximate top of foundation
Contour interval: .5 foot (elevation M.S.



MONOLITH 23

SCALE 1:5

NOTE: For legend see Monolith 1

... foundation

... foot (elevation M.S.L.)

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FOUNDATION REPORT
CONCRETE DAM
MONOLITH 23

RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE			PLATE
F			41

SCALE AS SHOWN

SHEET

REVISIONS

41

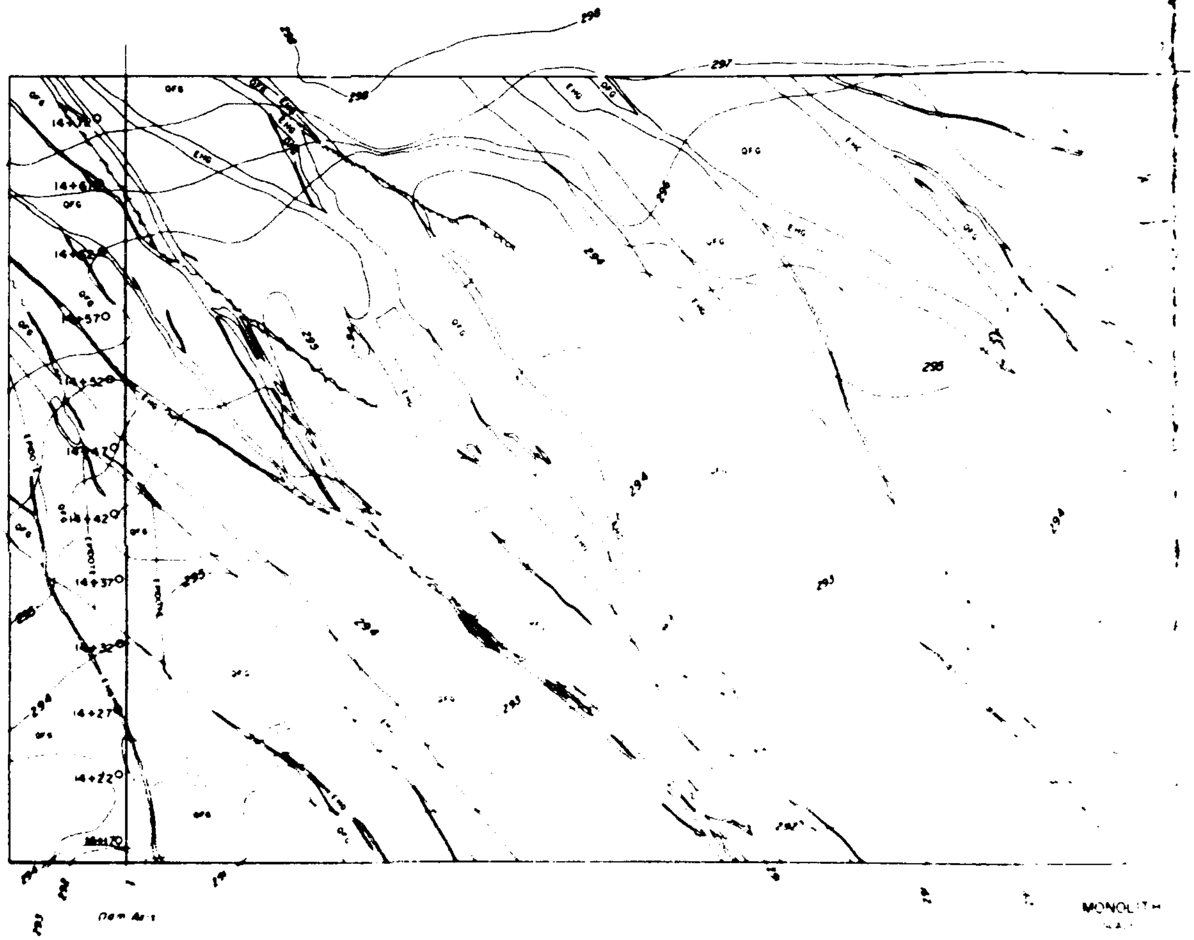
5 4

D

C

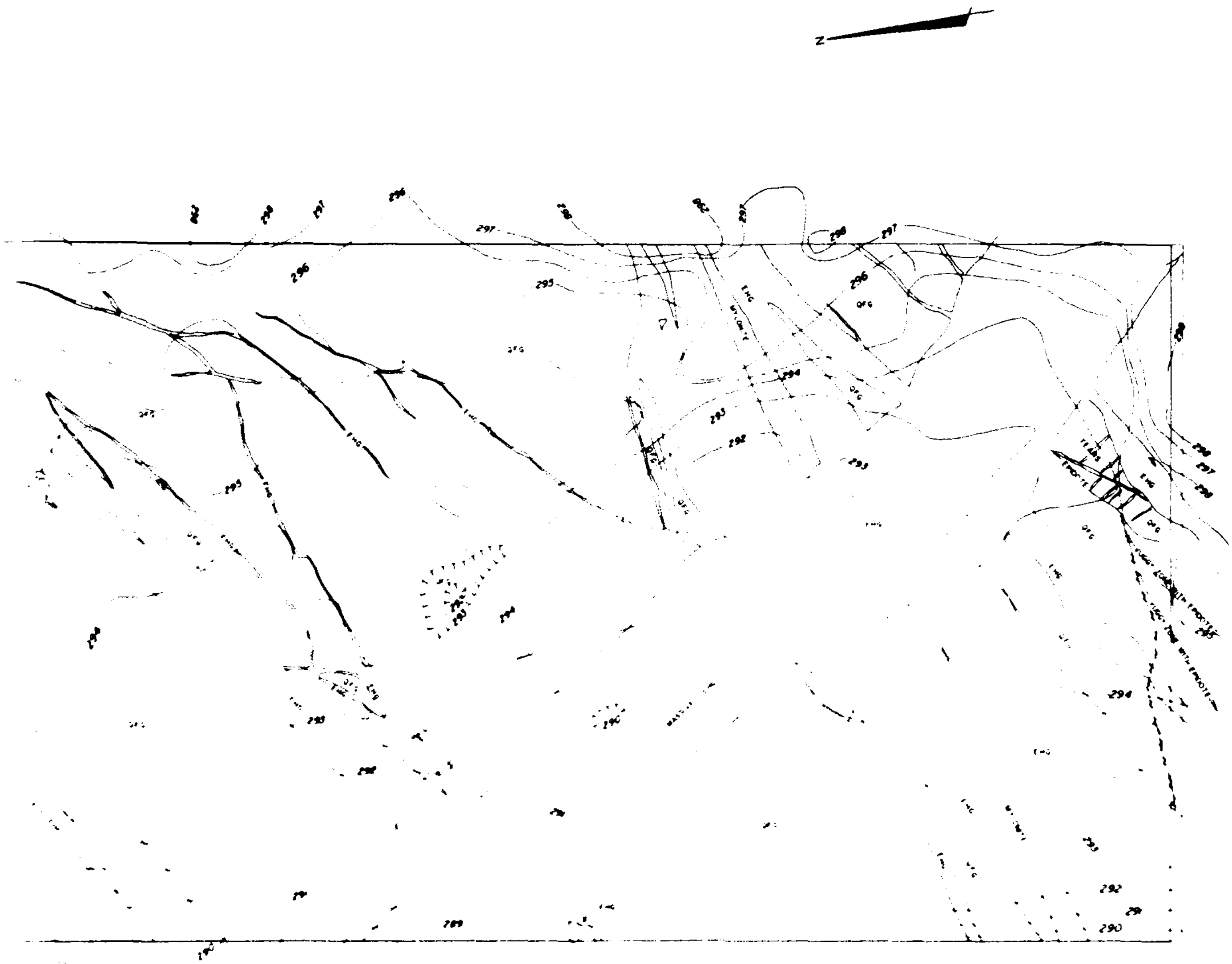
B

A



293 Approximate top of foundation
Contour interval 1.0 foot elevation M.S.L.

5 4



MONOLITH 24

SCALE 1" = 5'

NOTE: For legend see Monolith

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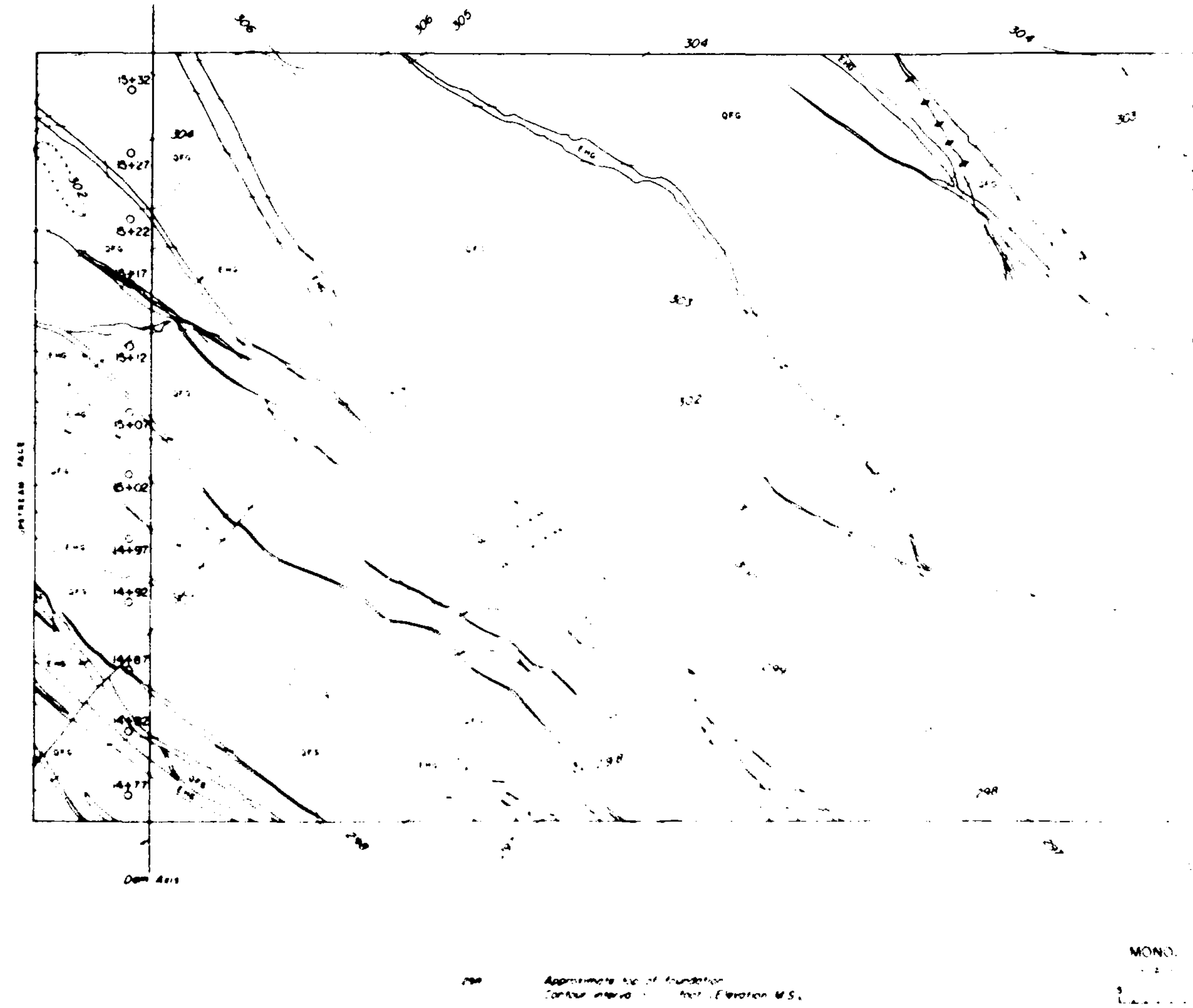
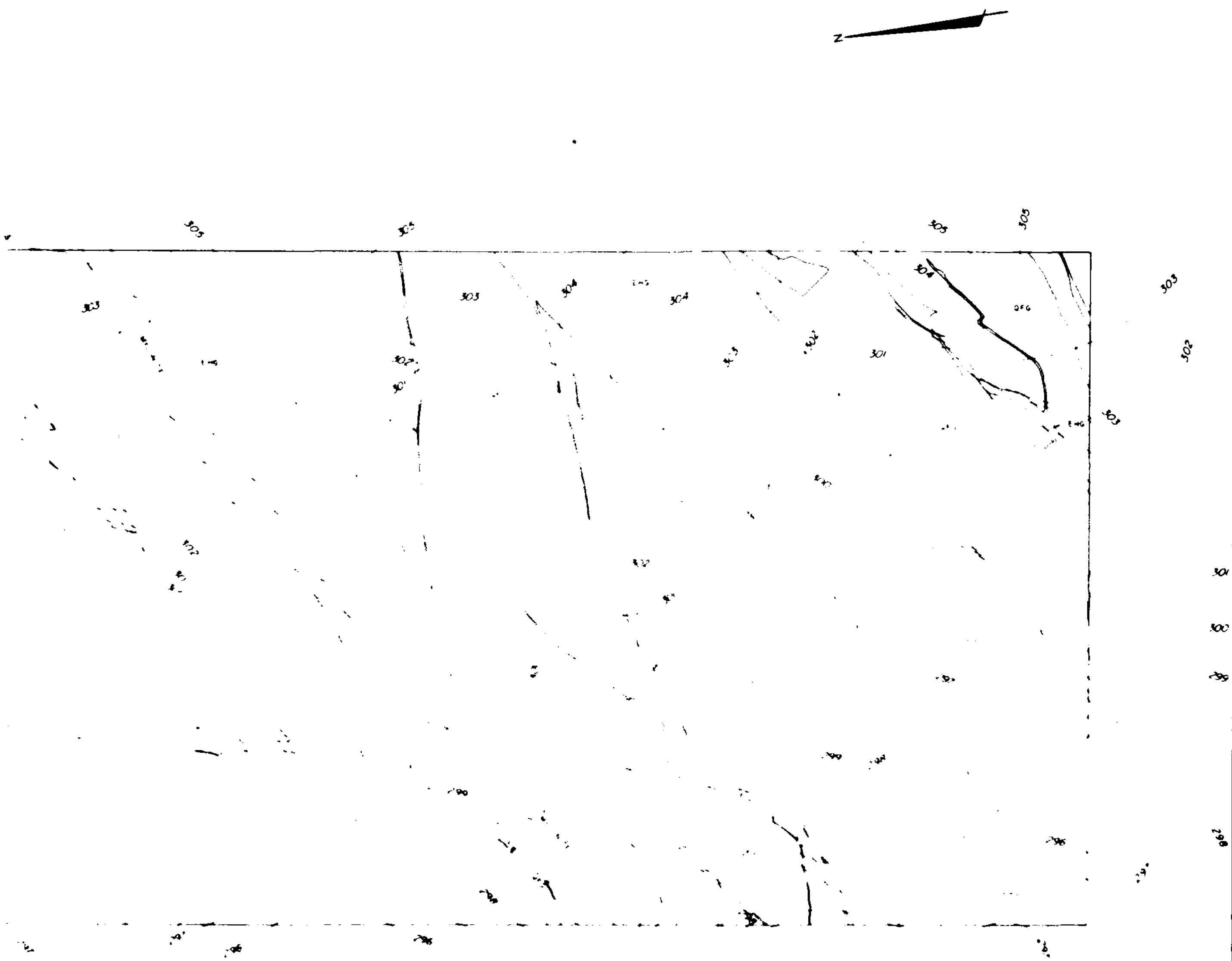
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
MONOLITH 24
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER — GEORGIA AND SOUTH CAROLINA

PLATE 42



306
305
304
303
302
298
15+32
15+27
15+22
15+17
15+12
15+07
15+02
14+97
14+92
14+87
14+82
14+77
QFG
FHG
Dam Axis
MONO.



MONOLITH 25

NOTE

For legend see ...

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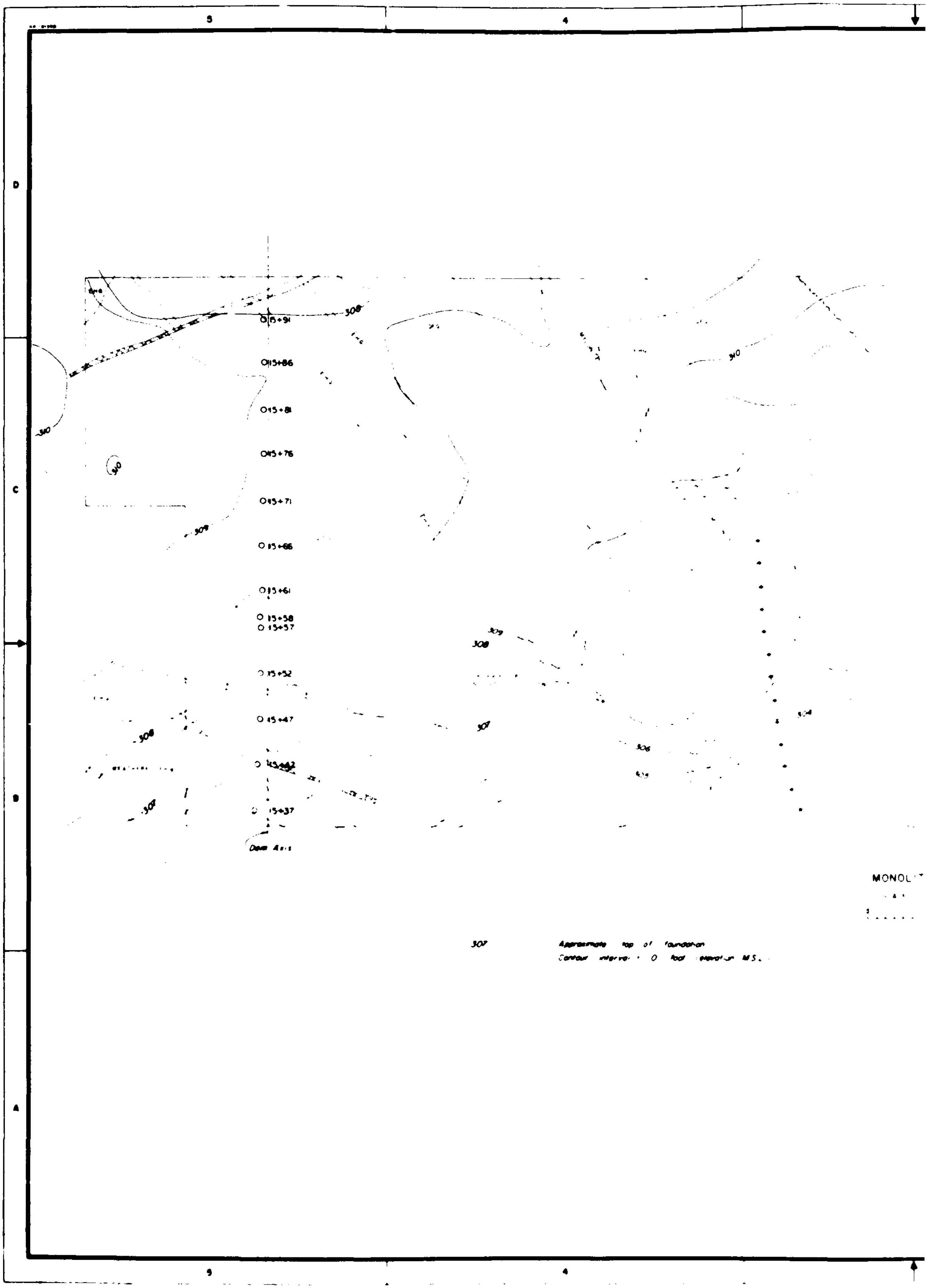
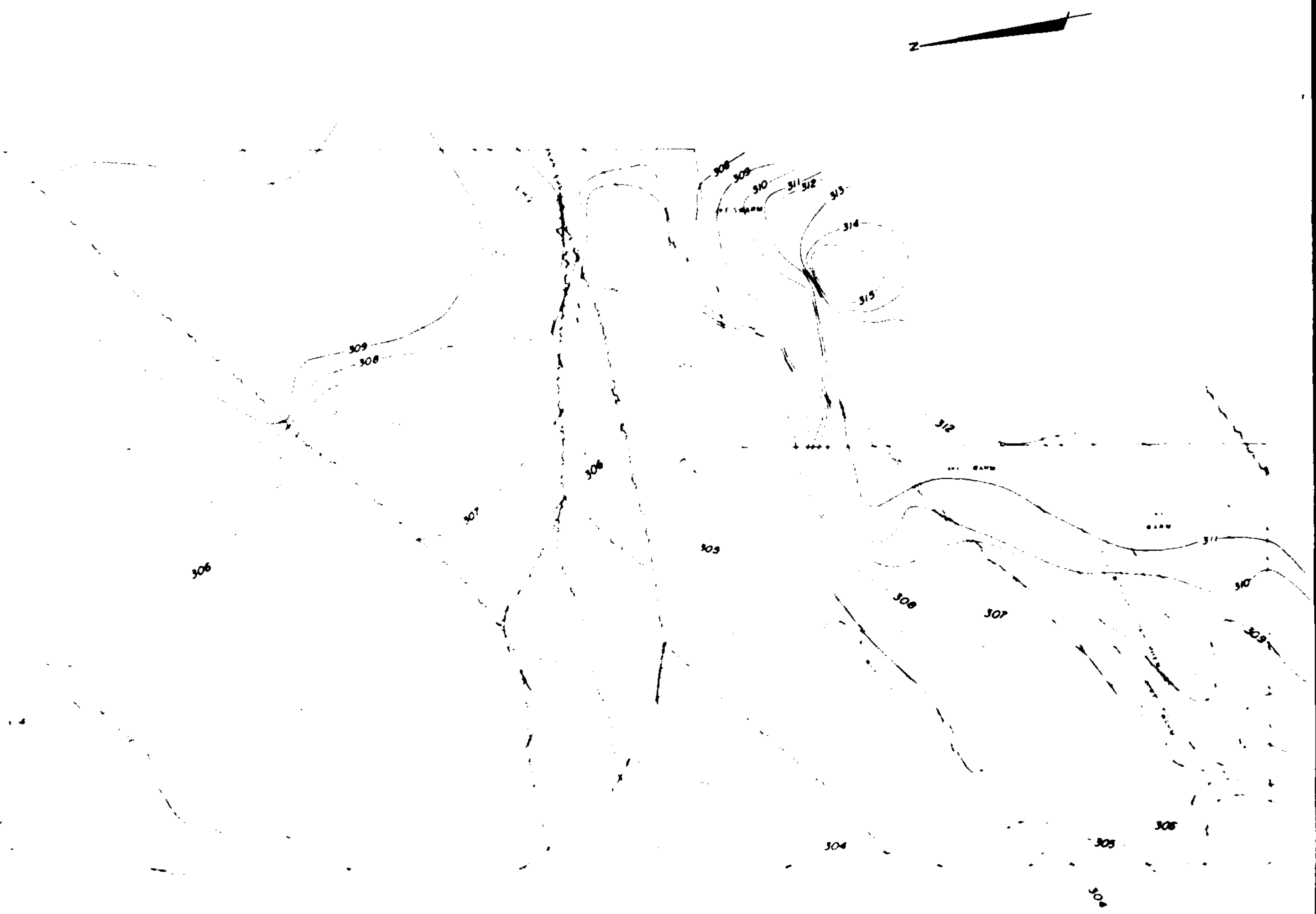
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 25
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 43



15+91
15+86
15+81
15+76
15+71
15+66
15+61
15+58
15+57
15+52
15+47
15+42
15+37
Dam Axis
308
310
309
307
306
304
307 Approximate top of foundation



MONOLITH 26

SCALE

NOTE For legend see Monolith

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FOUNDATION REPORT
CONCRETE DAM
MONOLITH 26
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

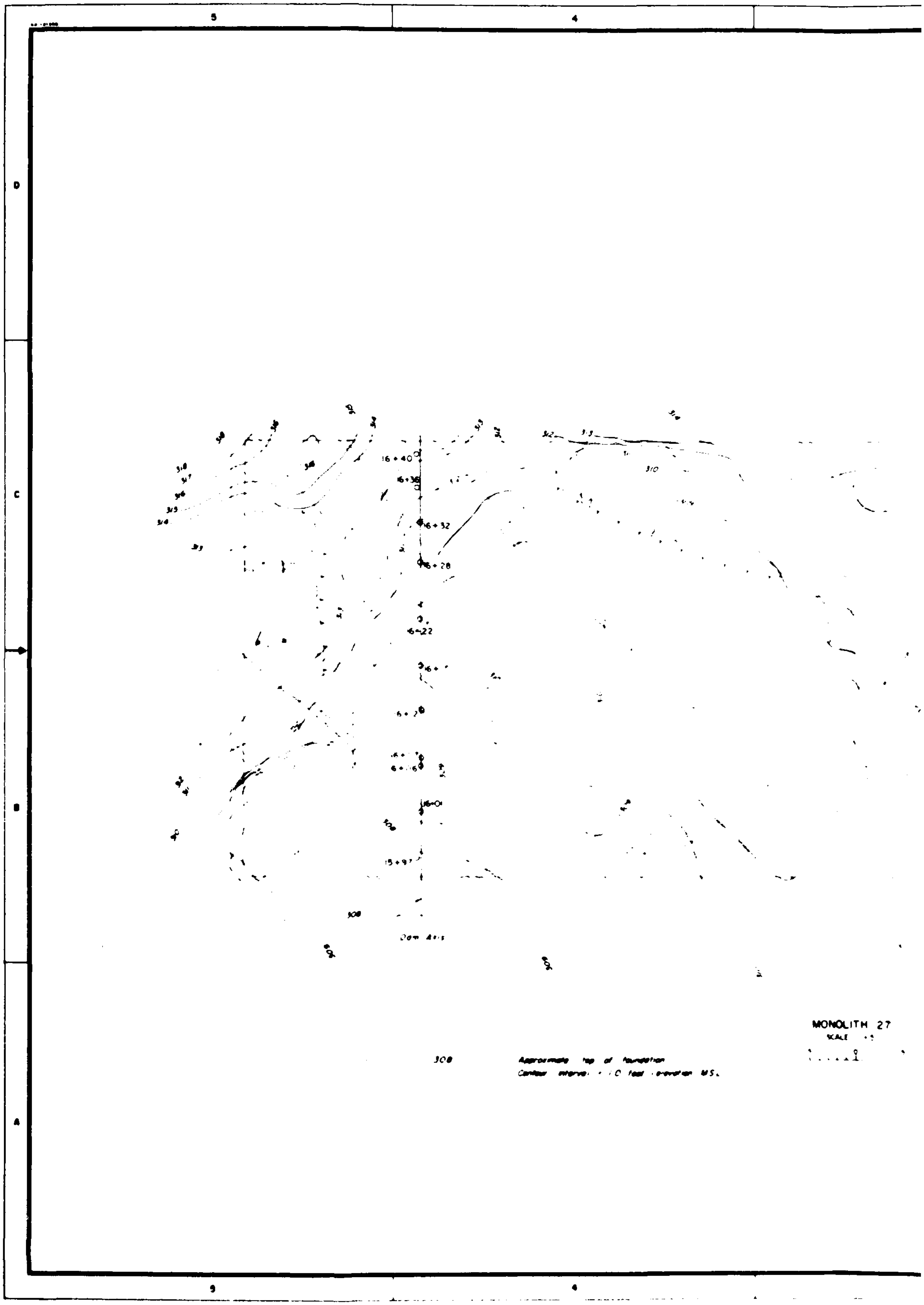
SIZE		DRAWING NO	PLATE
F			44

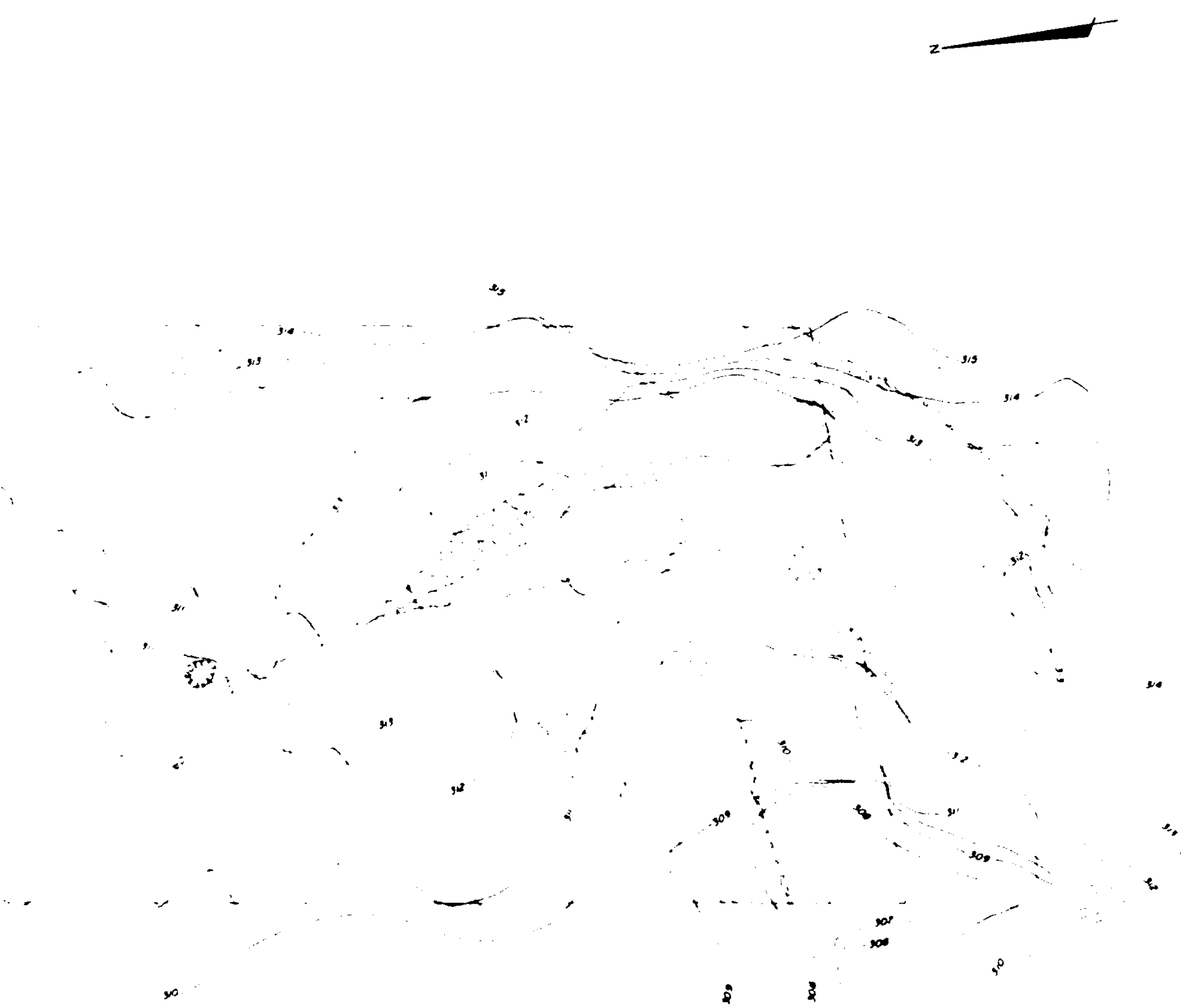
SCALE AS SHOWN | SHEET

REVISIONS

44

2


5
4
D
C
B
A
16+40
16+36
16+32
16+28
16+22
15+97
Dam Axis
MONOLITH 27
308 Approximate top of foundation
Contour interval 1.0 foot elevation MSL



MONOLITH 27

NOTE For legend see Monolith 1

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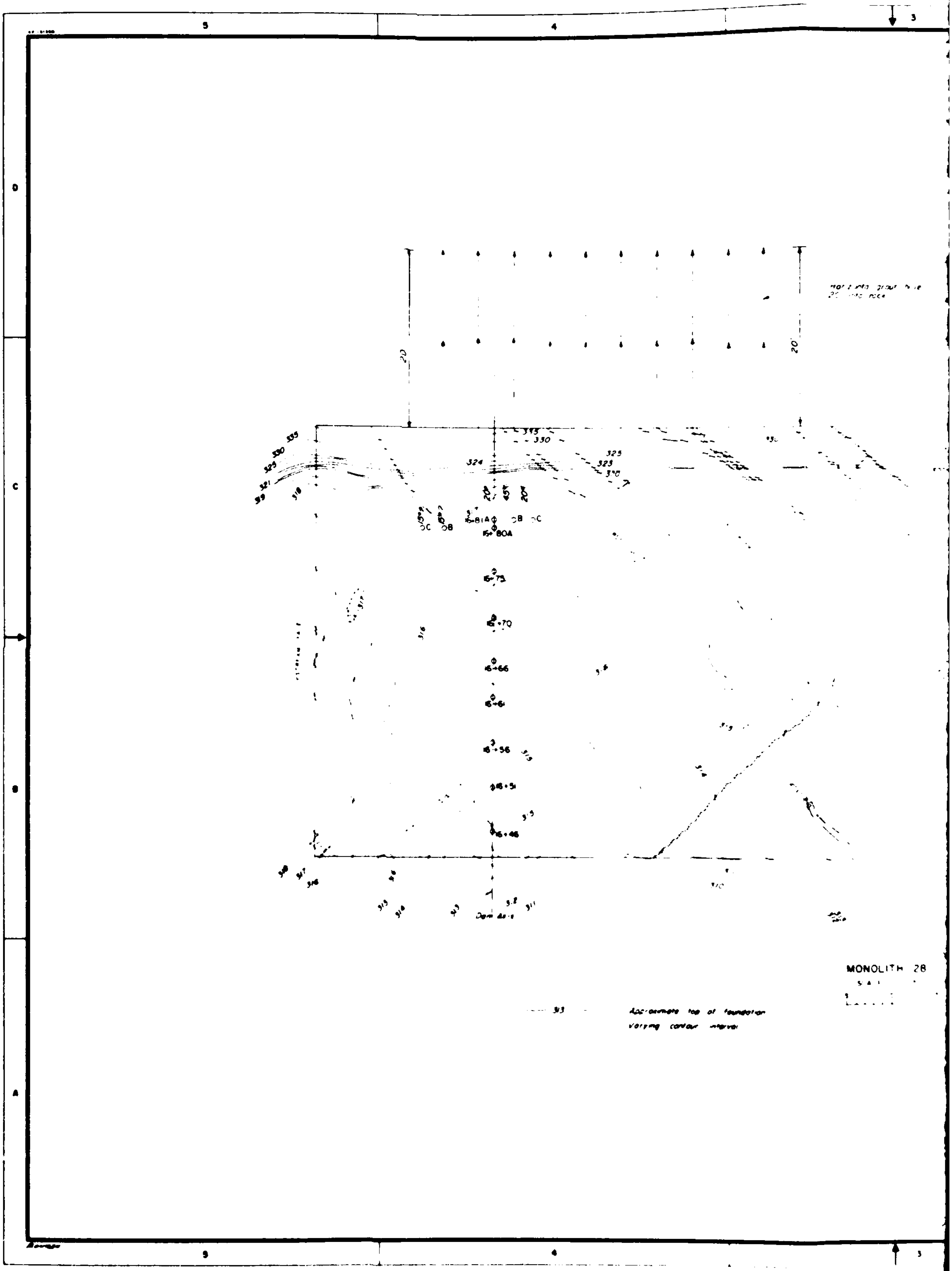
FOUNDATION REPORT
CONCRETE DAM
MONOLITH 27
RICHARD B RUSSELL PROJECT

SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE			PLATE
F			45

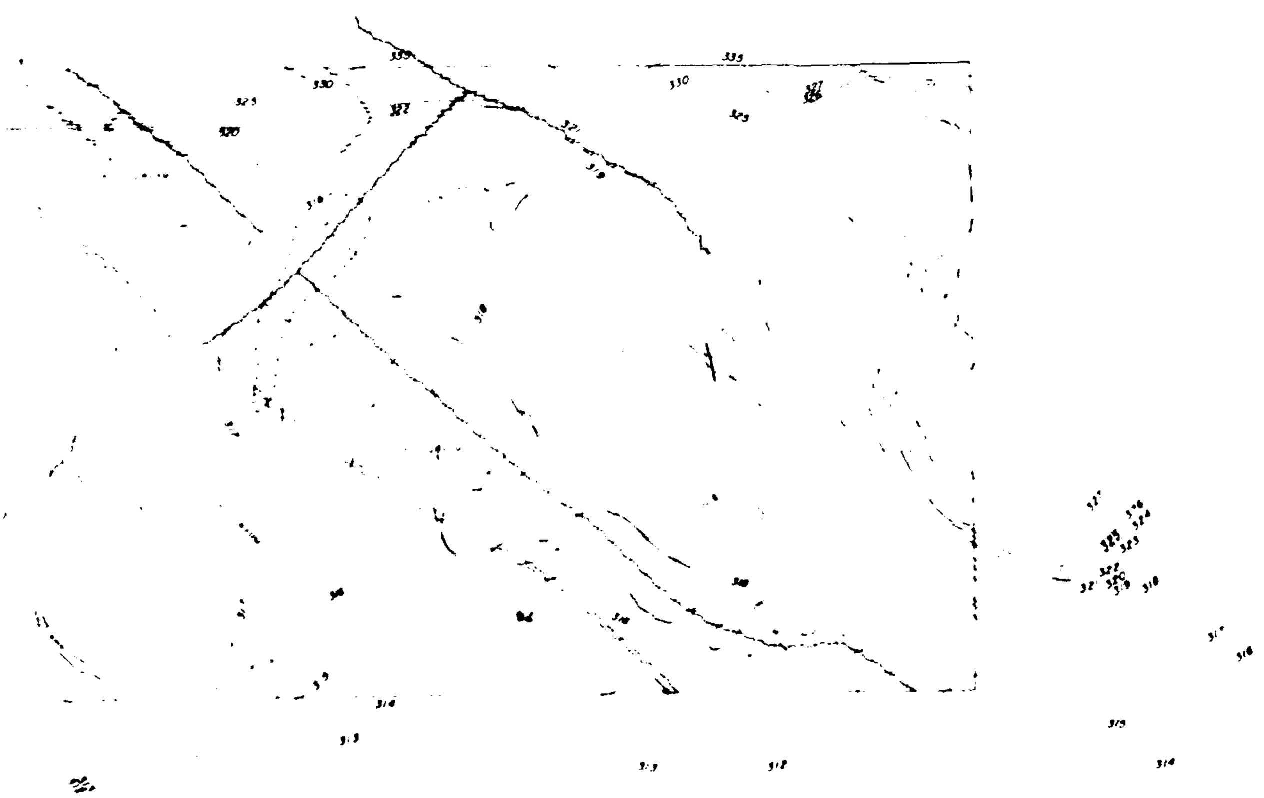
SCALE AS SHOWN

45



Horiz into grout hole 20' into rock
20'
20'
335
330
325
321
319
318
324
313
330
325
323
320
15+81A
15+80A
15+75
15+70
15+66
15+61
15+56
15+51
15+46
Dam Axis
313
Approximate top of foundation
Varying contour interval
MONOLITH 28

Horizontal grout hole
20' into rock



MONOLITH 28

SCALE 1" = 5'

NOTE For legend see Monolith 1

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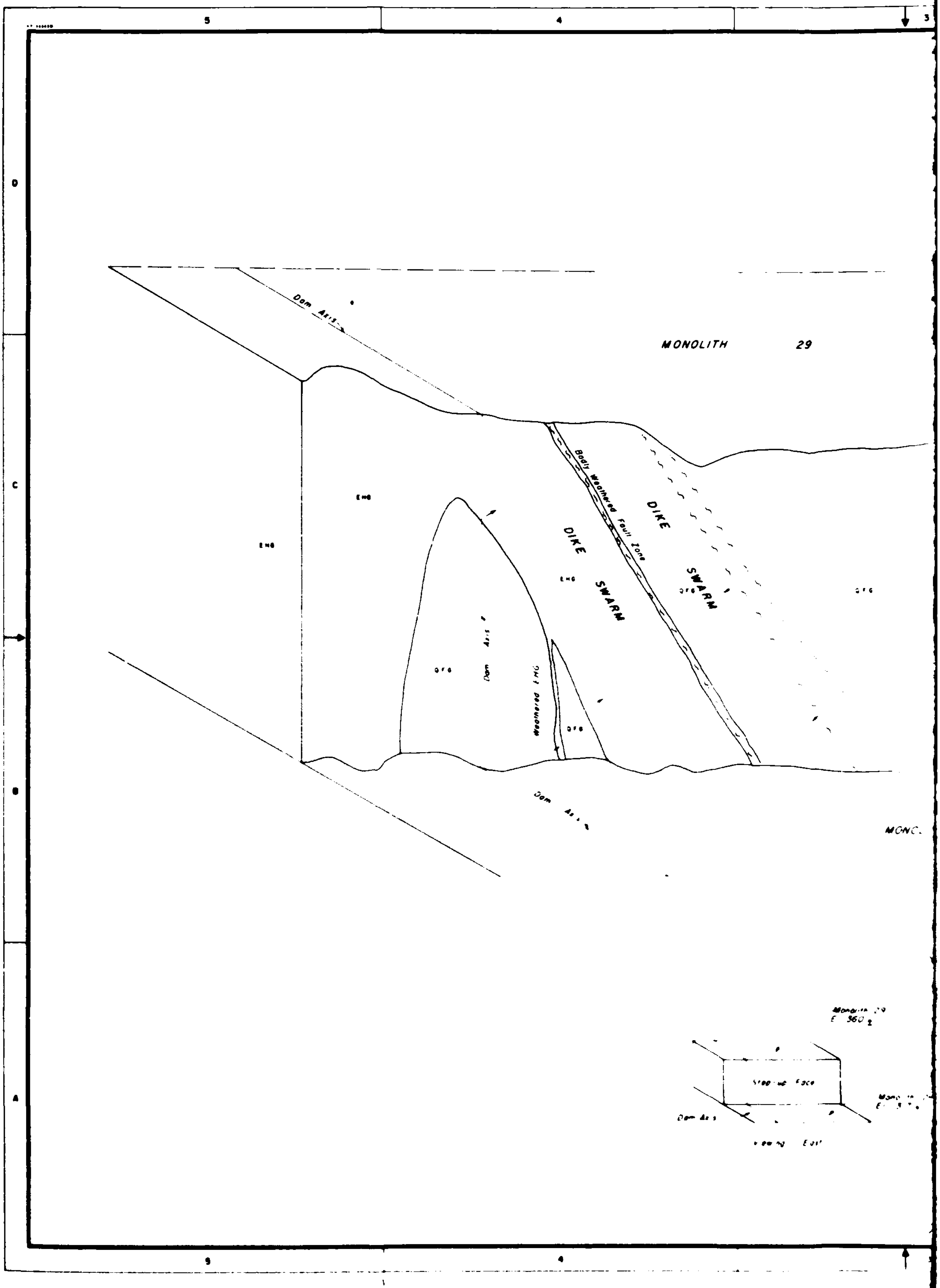
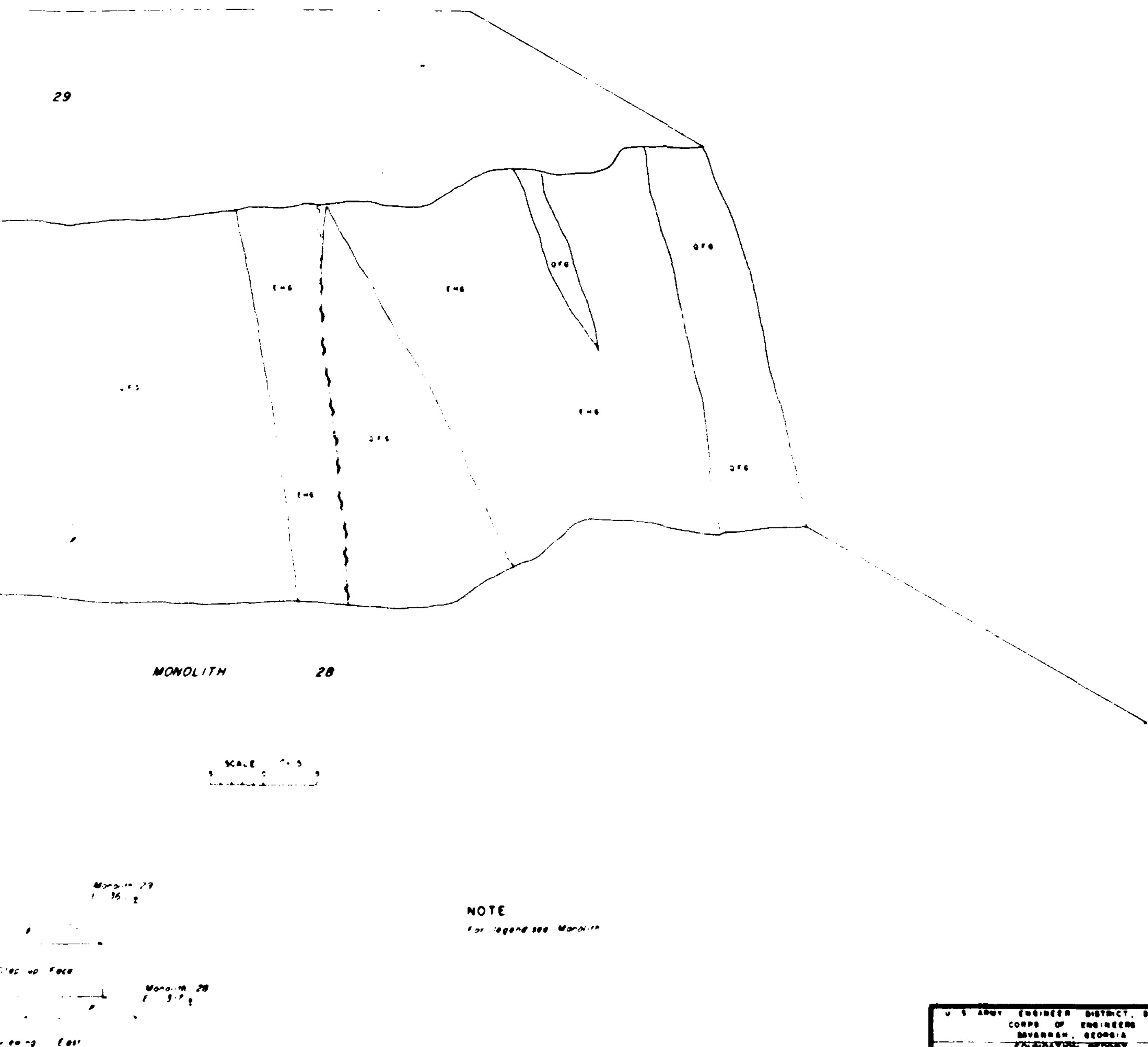
CONCRETE DAM
MONOLITH 28
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 46

SCALE AS SHOWN


5
4
D
C
B
A
Dam Axis
MONOLITH 29
Badly Weathered Fault Zone
DIKE SWARM
DIKE SWARM
EHG
EHG
EHG
QFG
QFG
QFG
QFG
Dam Axis
Weathered EHG
Dam Axis
MONO
Step-up Face
Dam Axis
Viewing East



NOTE

For legend see Monolith

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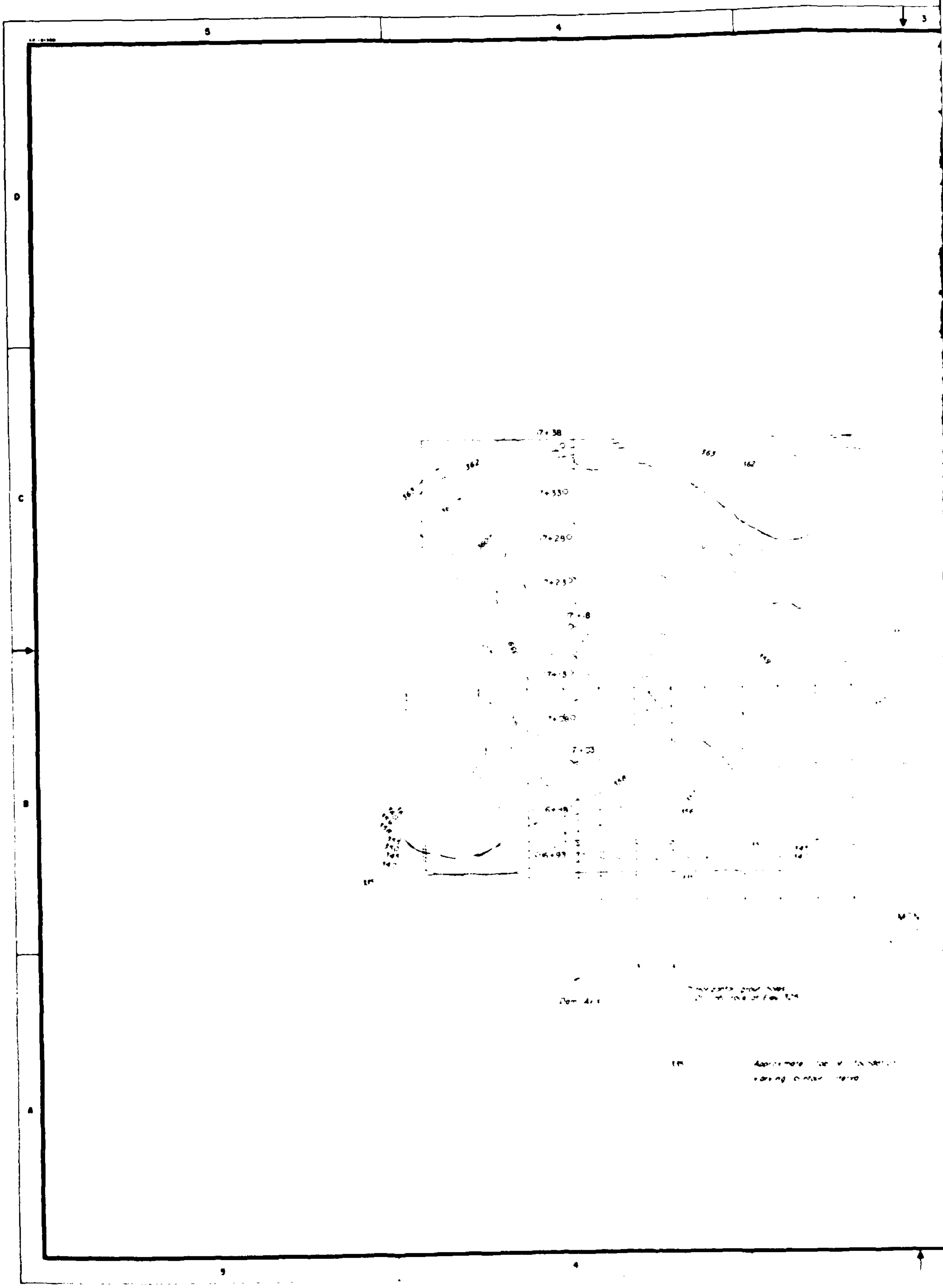
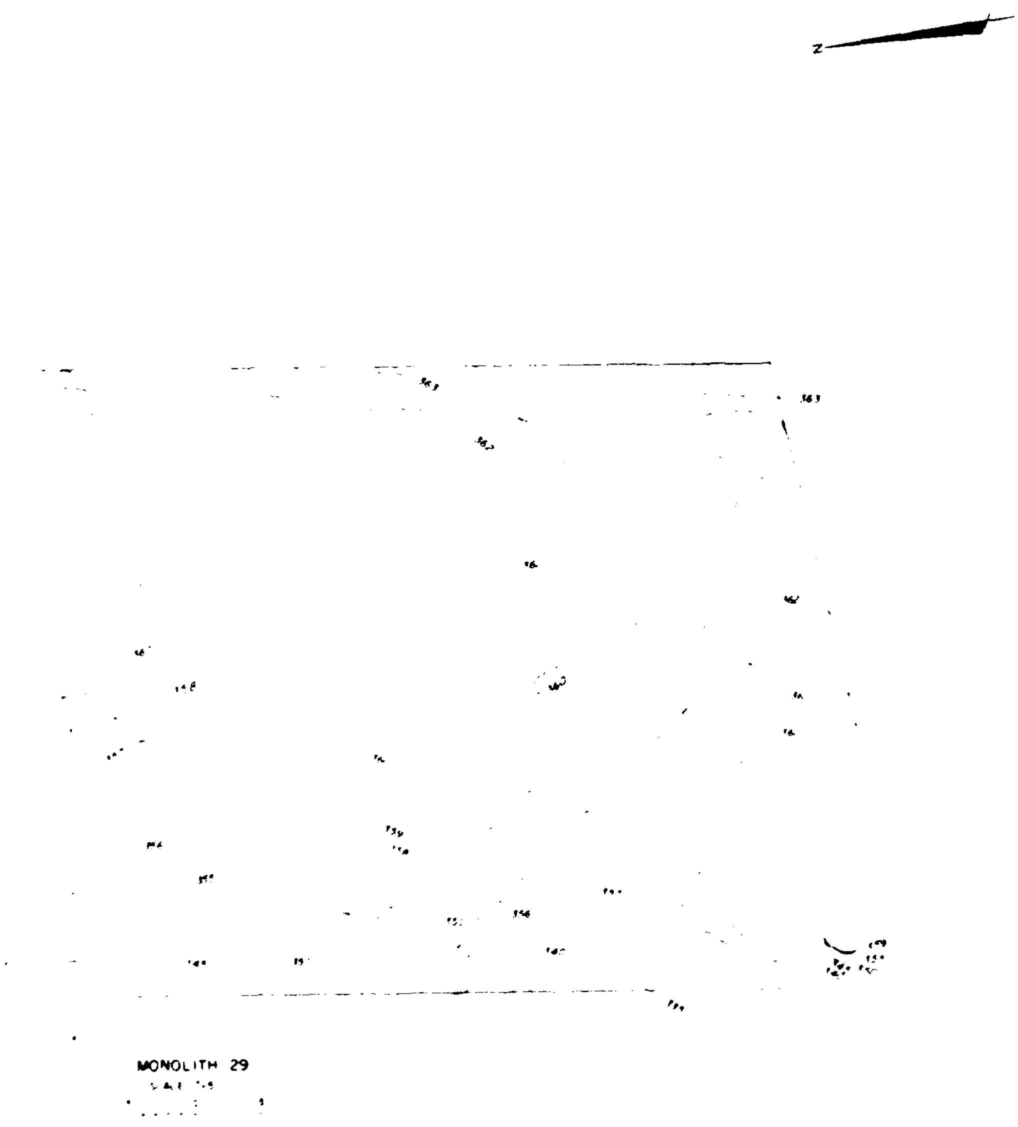
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
GEOLOGY OF THE STEP-UP FACE
BETWEEN MONOLITHS 28 & 29
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 47


5
4
3
D
C
B
A
7+38
362
363



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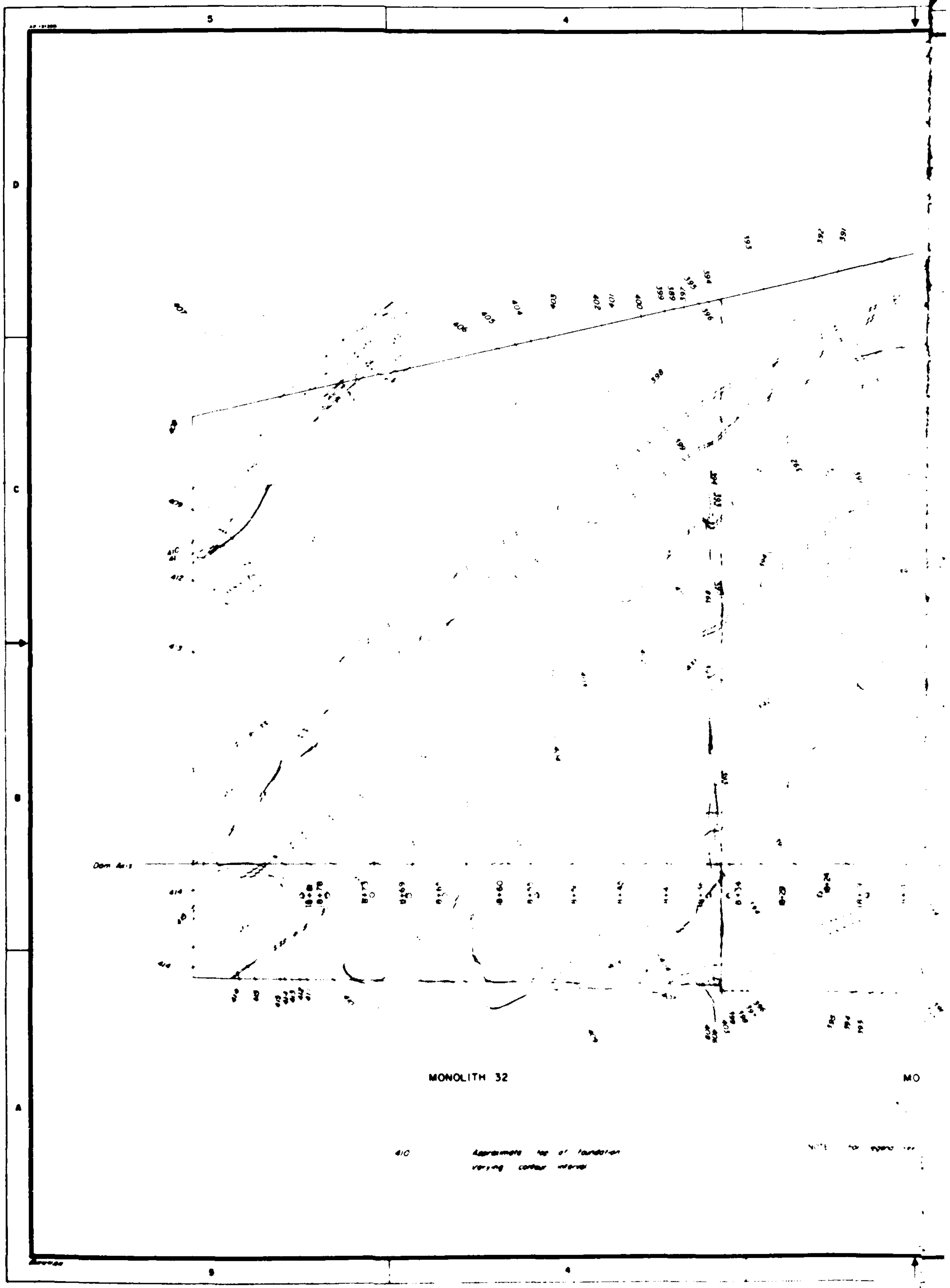
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
MONOLITH 29
RICHARD B RUSSELL PROJECT

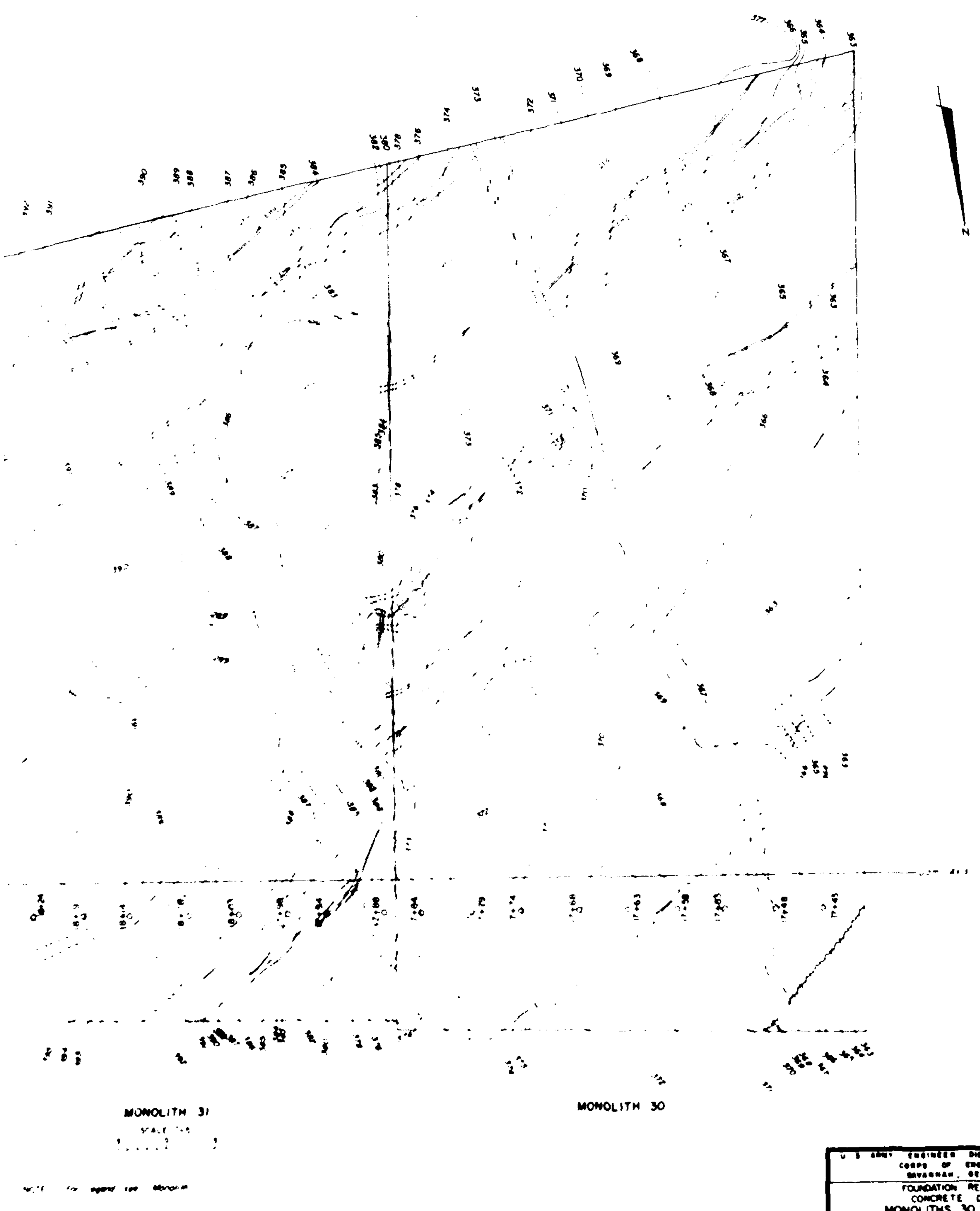
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 48

REVISIONS



Dam Axis
MONOLITH 32
MO
410
Approximate top of foundation
varying contour interval



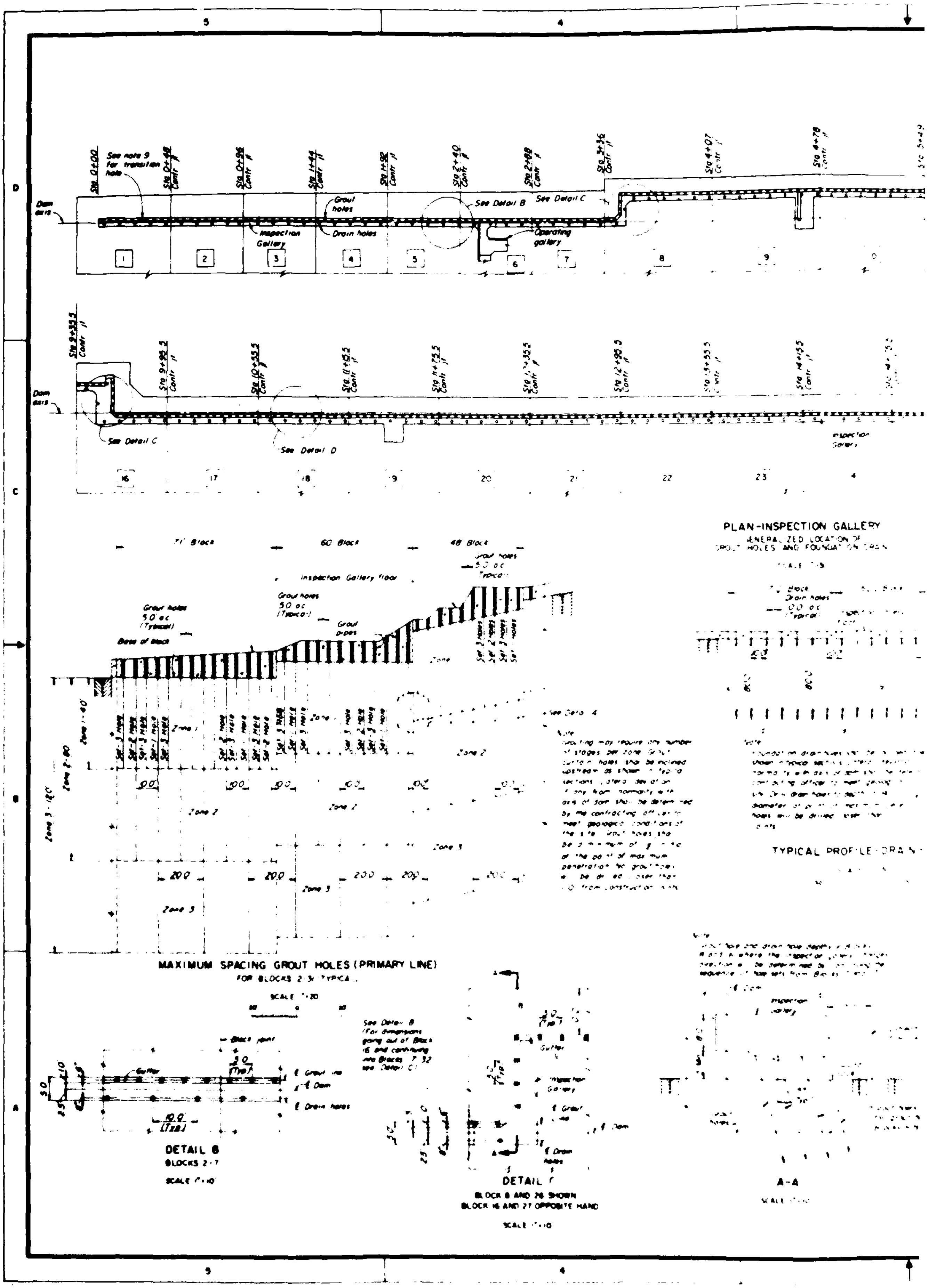
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
MONOLITHS 30, 31, & 32
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

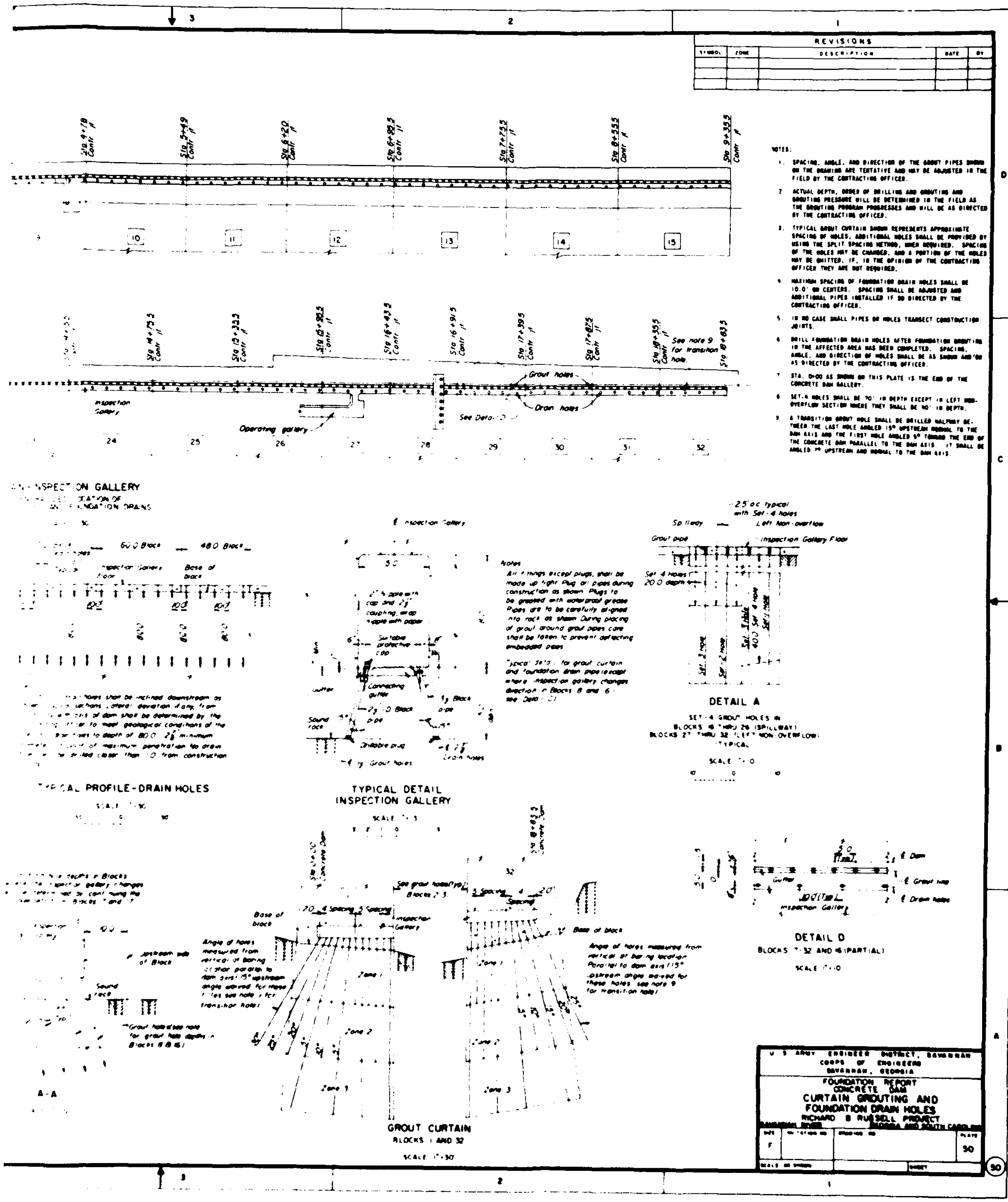
PLATE 49

SCALE AS SHOWN

49



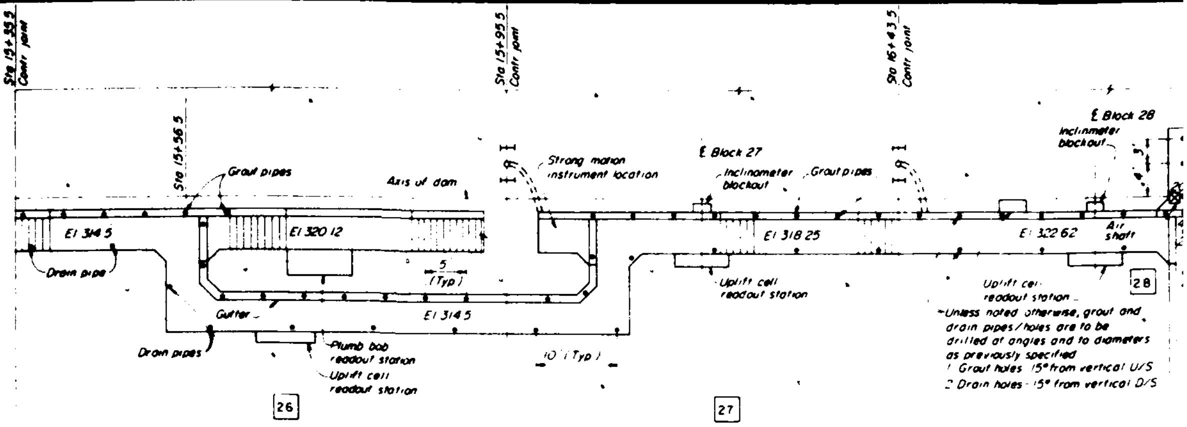
See note 9 for transition hole
Sta 0+00
Grout holes
Inspection Gallery
Drain holes
See Detail B
See Detail C
Operating gallery
Dam axis
See Detail C
See Detail D
PLAN-INSPECTION GALLERY
71' Block
60' Block
48' Block
Inspection Gallery floor
Grout holes 5.0 oc (Typical)
Grout pipes
Base of block
Zone 1
Zone 2
Zone 3
MAXIMUM SPACING GROUT HOLES (PRIMARY LINE)
TYPICAL PROFILE-DRAIN
DETAIL B
BLOCKS 2-7
Gutter
Block joint
DETAIL C
BLOCK 8 AND 26 SHOWN
BLOCK 16 AND 27 OPPOSITE HAND
A-A



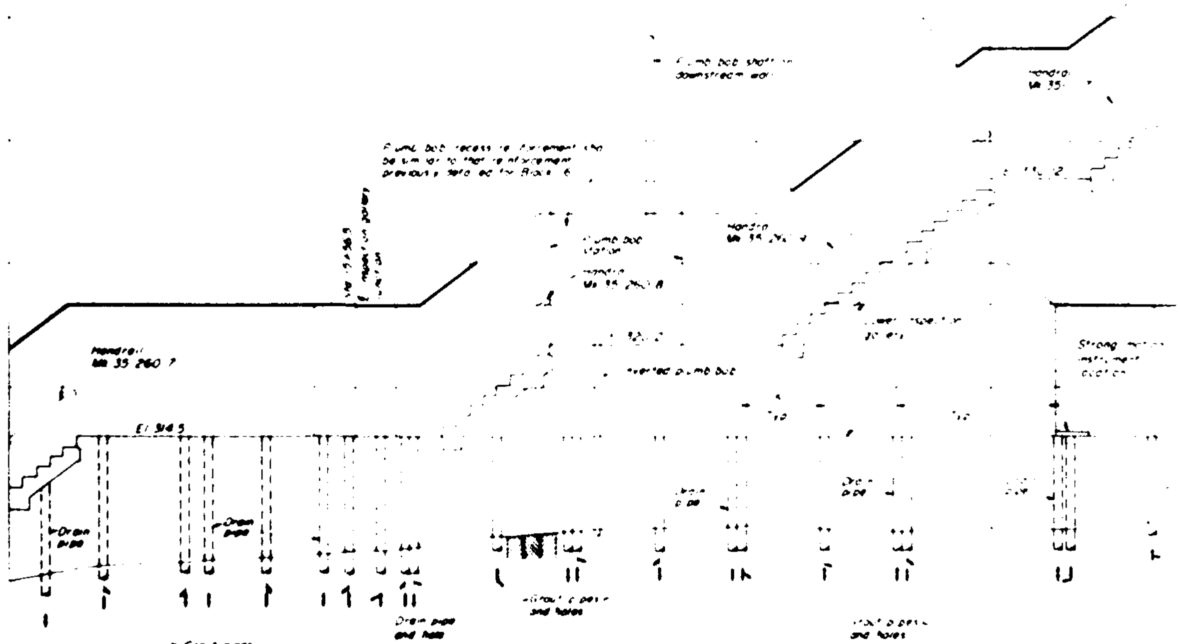
NOTES:

1. SPACING, ANGLE, AND DIRECTION OF THE GROUT PIPES SHOWN ON THE DRAWING ARE TENTATIVE AND MAY BE ADJUSTED IN THE FIELD BY THE CONTRACTING OFFICER.
2. ACTUAL DEPTH, ORDER OF DRILLING AND GROUTING AND GROUTING PRESSURE WILL BE DETERMINED IN THE FIELD AS THE GROUTING PROGRAM PROGRESSES AND WILL BE AS DIRECTED BY THE CONTRACTING OFFICER.
3. TYPICAL GROUT CURTAIN SHOWN REPRESENTS APPROXIMATE SPACING OF HOLES, ADDITIONAL HOLES SHALL BE PROVIDED BY USING THE SPLIT SPACING METHOD, WHEN REQUIRED. SPACING OF THE HOLES MAY BE CHANGED, AND A PORTION OF THE HOLES MAY BE OMITTED, IF, IN THE OPINION OF THE CONTRACTING OFFICER THEY ARE NOT REQUIRED.
4. MAXIMUM SPACING OF FOUNDATION DRAIN HOLES SHALL BE 10.0' ON CENTERS. SPACING SHALL BE ADJUSTED AND ADDITIONAL PIPES INSTALLED IF SO DIRECTED BY THE CONTRACTING OFFICER.
5. IN NO CASE SHALL PIPES OR HOLES TRANSECT CONSTRUCTION JOINTS.
6. DRILL FOUNDATION DRAIN HOLES AFTER FOUNDATION GROUTING IN THE AFFECTED AREA HAS BEEN COMPLETED. SPACING, ANGLE, AND DIRECTION OF HOLES SHALL BE AS SHOWN AND/OR AS DIRECTED BY THE CONTRACTING OFFICER.
7. STA. 0+00 AS SHOWN ON THIS PLATE IS THE END OF THE CONCRETE DAM GALLERY.
8. SET-4 HOLES SHALL BE 70' IN DEPTH EXCEPT IN LEFT NON-OVERFLOW SECTION WHERE THEY SHALL BE 40' IN DEPTH.
9. A TRANSITION GROUT HOLE SHALL BE DRILLED HALFWAY BETWEEN THE LAST HOLE ANGLED 15° UPSTREAM NORMAL TO THE DAM AXIS AND THE FIRST HOLE ANGLED 5° TOWARD THE END OF THE CONCRETE DAM PARALLEL TO THE DAM AXIS. IT SHALL BE ANGLED 7° UPSTREAM AND NORMAL TO THE DAM AXIS.

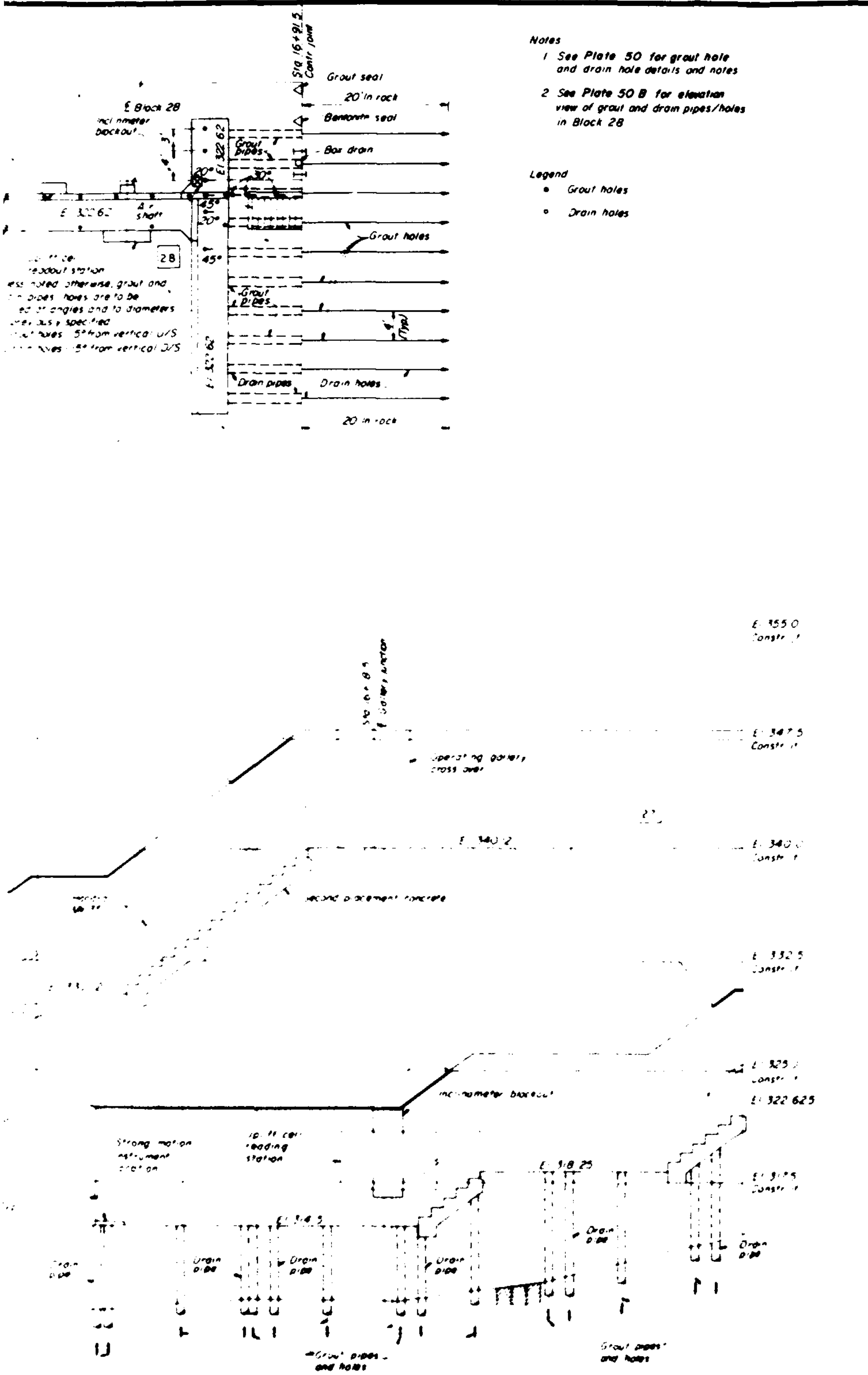
2



LOWER INSPECTION GALLERY
GENERALIZED LOCATION PLAN OF GROUT PIPES AND FOUNDATION DRAINS



LOWER INSPECTION GALLERY
GENERALIZED ELEVATION OF GROUT PIPES AND FOUNDATION DRAINS



Notes
1. See Plate 50 for grout hole and drain hole details and notes
2. See Plate 50 B for elevation view of grout and drain pipes/holes in Block 28

Legend
• Grout holes
∘ Drain holes

Note
Not all grout and drain pipes and holes are shown. See plan for locations of all pipes.

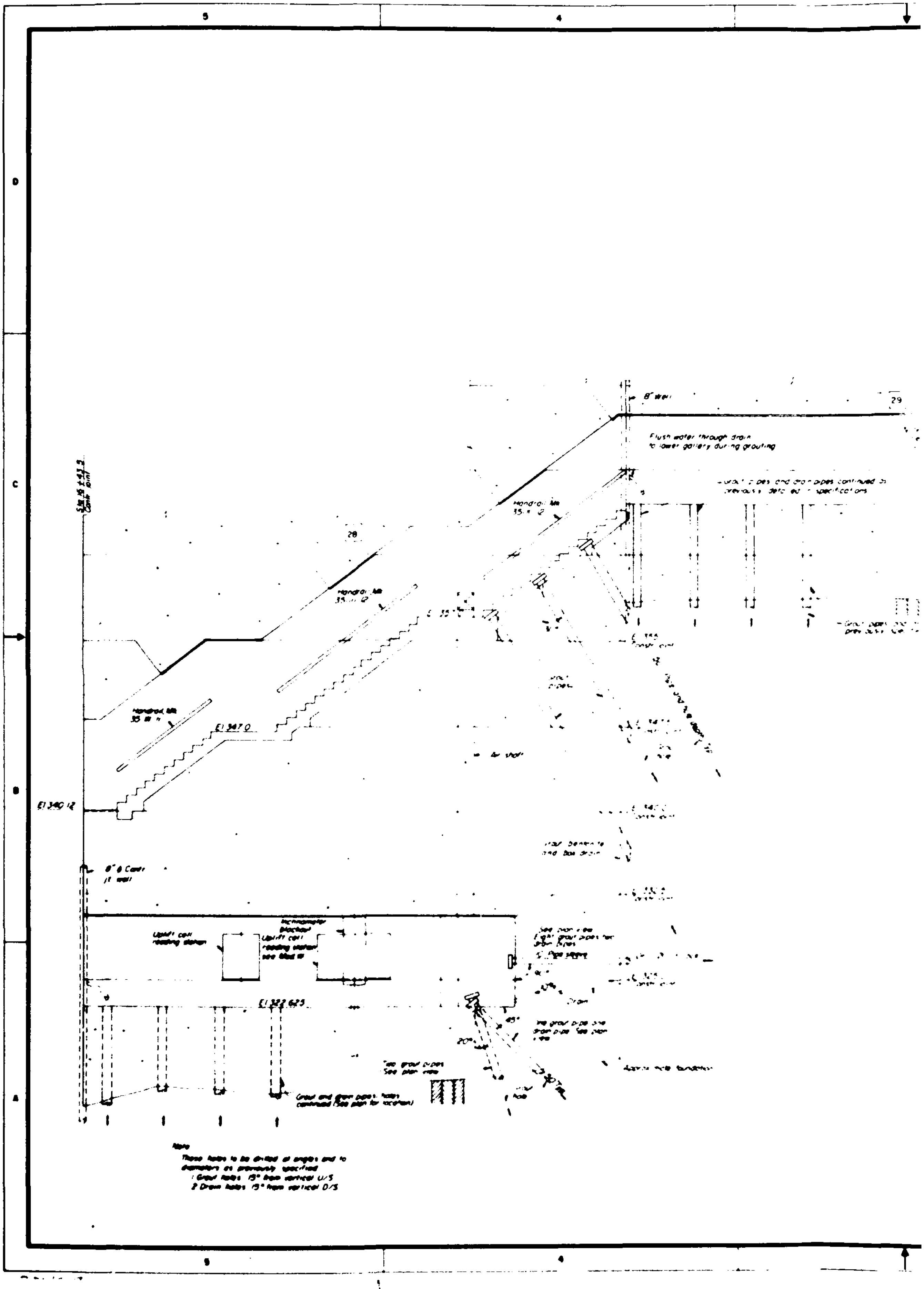
ELEVATION OF GROUT
FOUNDATION DRAINS

REVISIONS
SHEET ADDED BY MCR NO. 70 ... 12 OCT 79

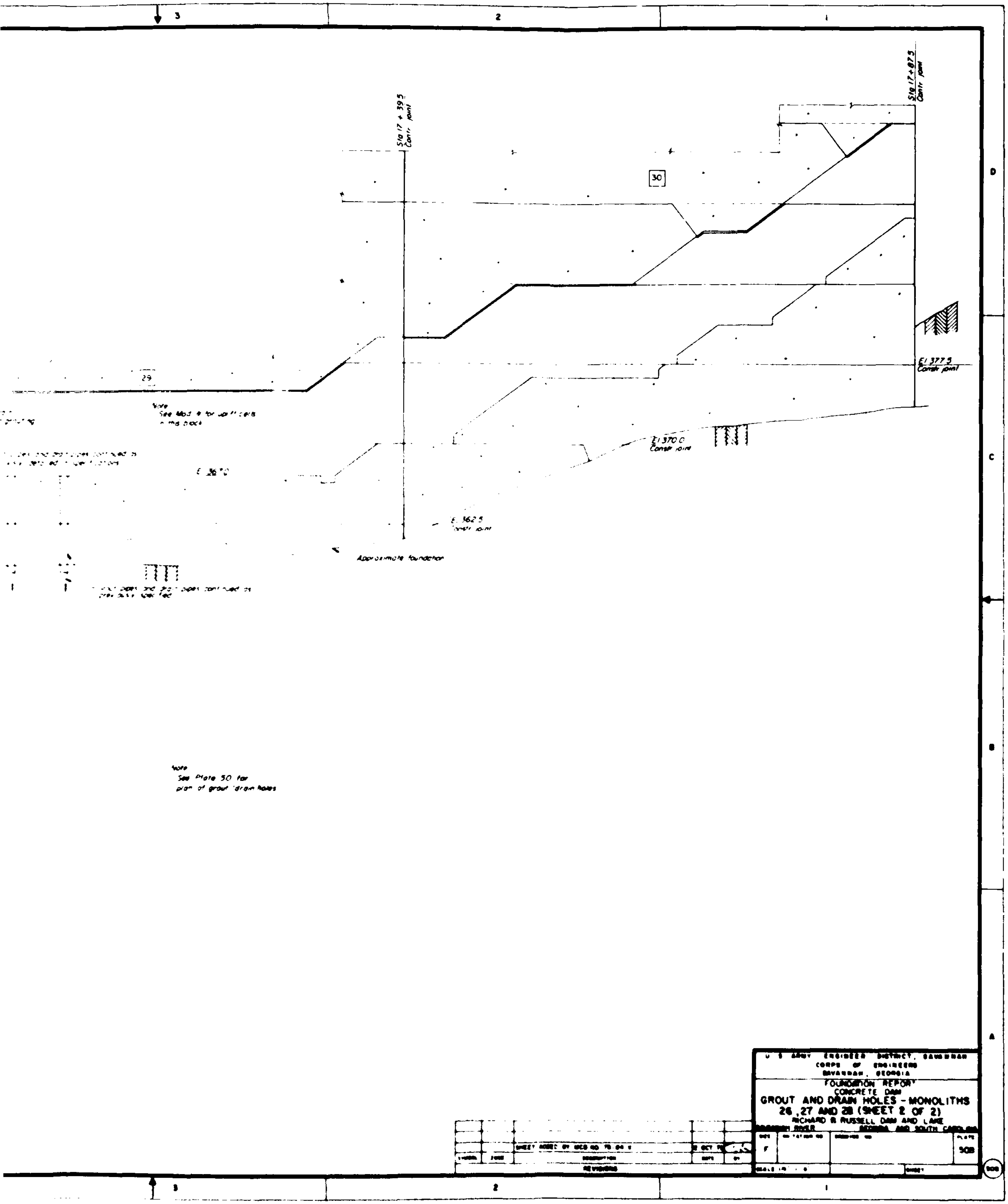
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
GROUT AND DRAIN HOLES - MONOLITHS 26, 27 AND 28 (SHEET 1 OF 2)
RICHARD B. RUSSELL DAM AND LAKE
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 50A
SCALE AS SHOWN

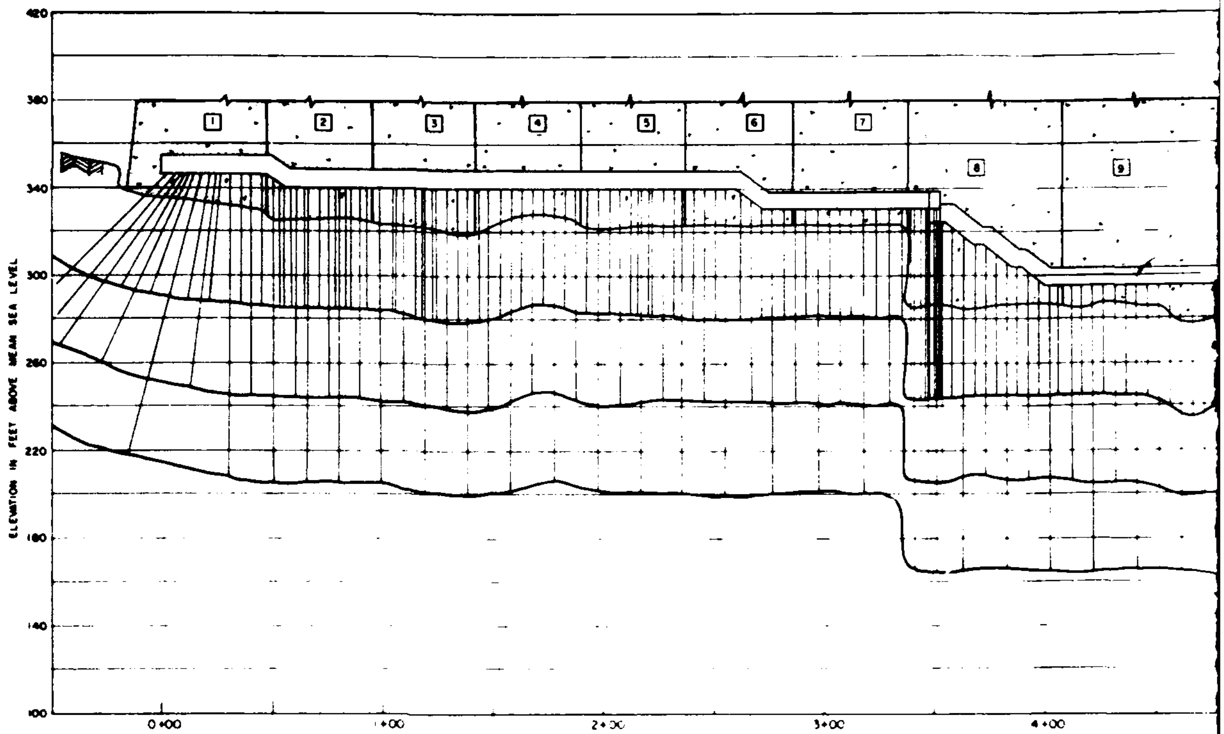


5
4
D
C
B
A
8" Wall
29
28
Flush water through drain to lower gallery during grouting
Handrail Mk
Handrail Mk 35
El 347.0
El 340.12
Air shaft
8" ø Cased jt wall
Uplift cell reading station
Inclinometer blockout
El 322.625
Grout and drain pipes, holes continued (See plan for location)
Two grout pipes See plan view
Drain
Note
These holes to be drilled at angles and to diameters as previously specified
1 Grout holes 15° from vertical U/S
2 Drain holes 15° from vertical D/S



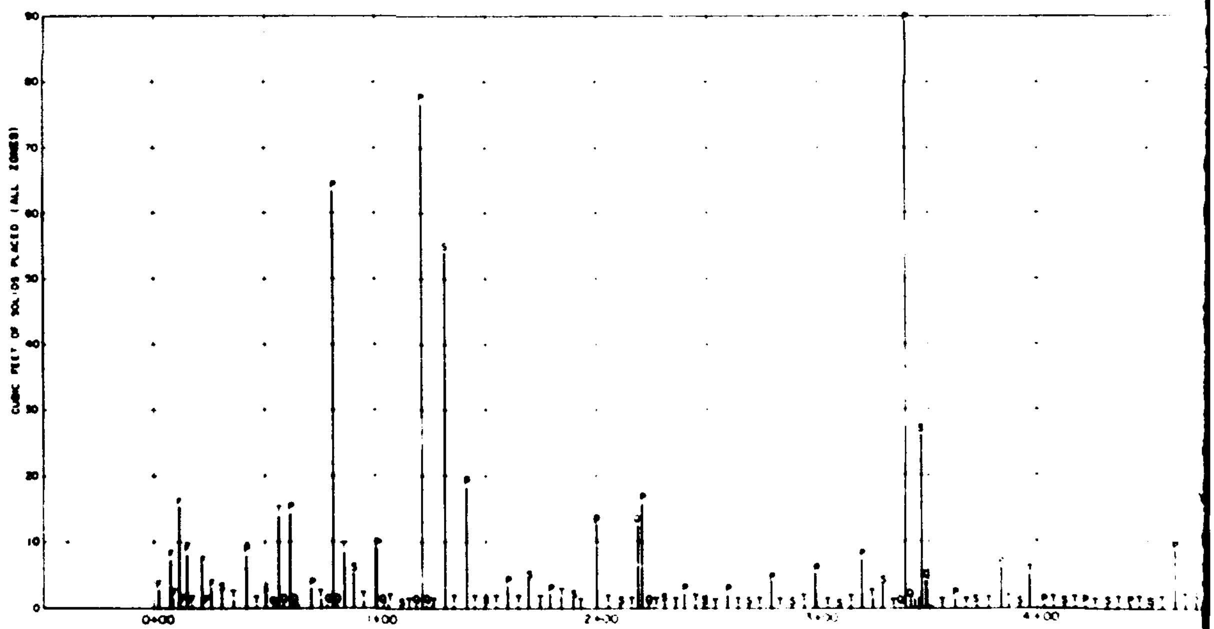
Sta 17 + 39.5
Contr joint
Sta 17+87.5
Contr joint
30
29
El 377.5
Constr joint
El 370.0
Constr joint
El 362.5
Constr joint
Approximate foundation
Note
See Plate 50 for
plan of grout/drain holes
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
CONCRETE DAM
GROUT AND DRAIN HOLES - MONOLITHS
26, 27 AND 28 (SHEET 2 OF 2)
RICHARD B RUSSELL DAM AND LAKE
REVISIONS
50B

2



CURTAIN GROUTING

SCALE



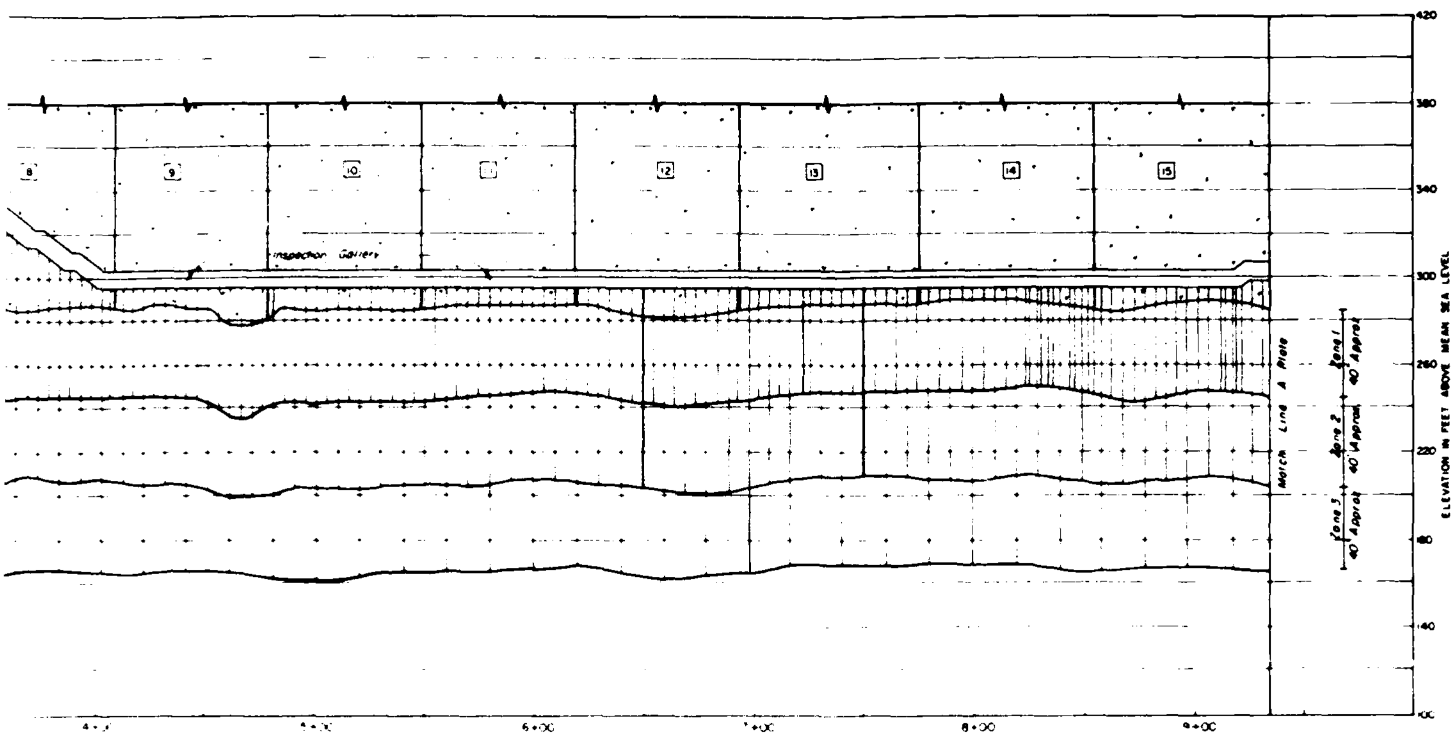
GROUT TAKES

LEGEND

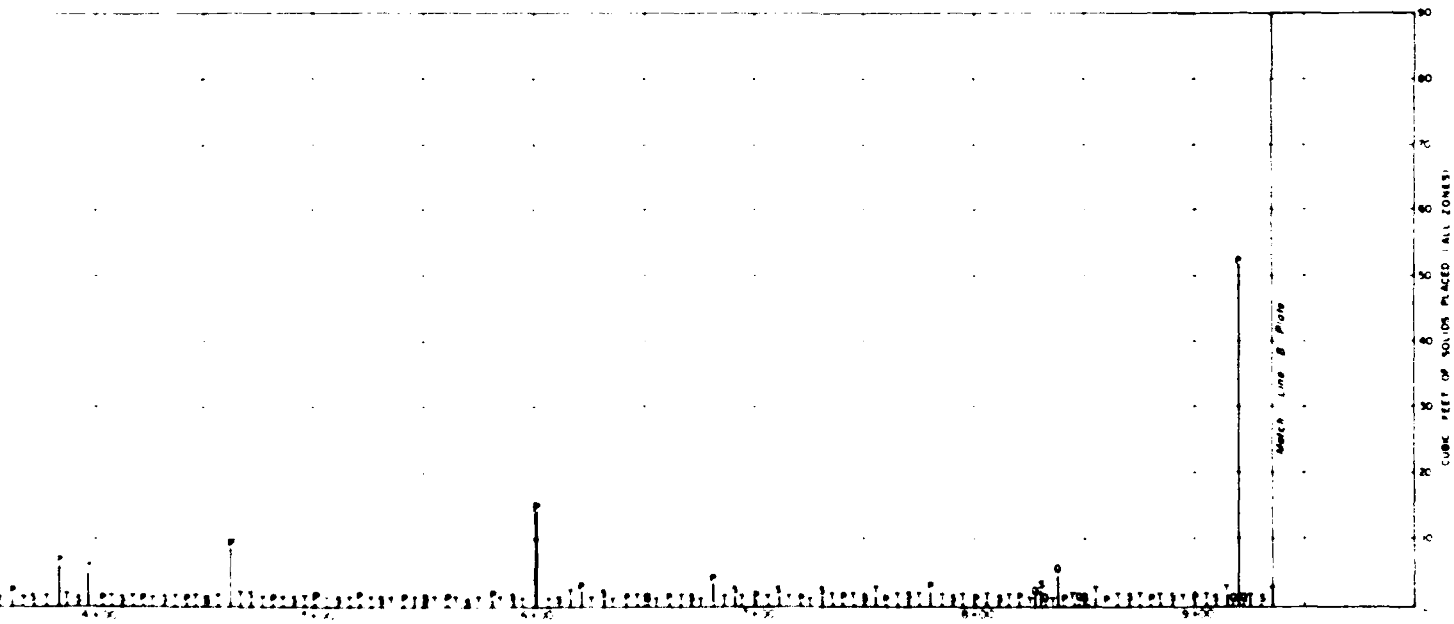
- P Primary grout hole
- S Secondary grout hole
- T Tertiary grout hole
- Q Quaternary grout hole
- F Fan grout hole

NOTES

1) Grout line is a single-line curtain
2) All holes (other than fan holes) are inclined in an upstream direction
3) Refer to Plate 50 for "As-designed" grouting schedule
4) Split-spacing of holes was determined by communication experienced between holes or by grout takes greater than bags per zone
5. For more data, refer to Section of text



CURTAIN GROUTING PROFILE

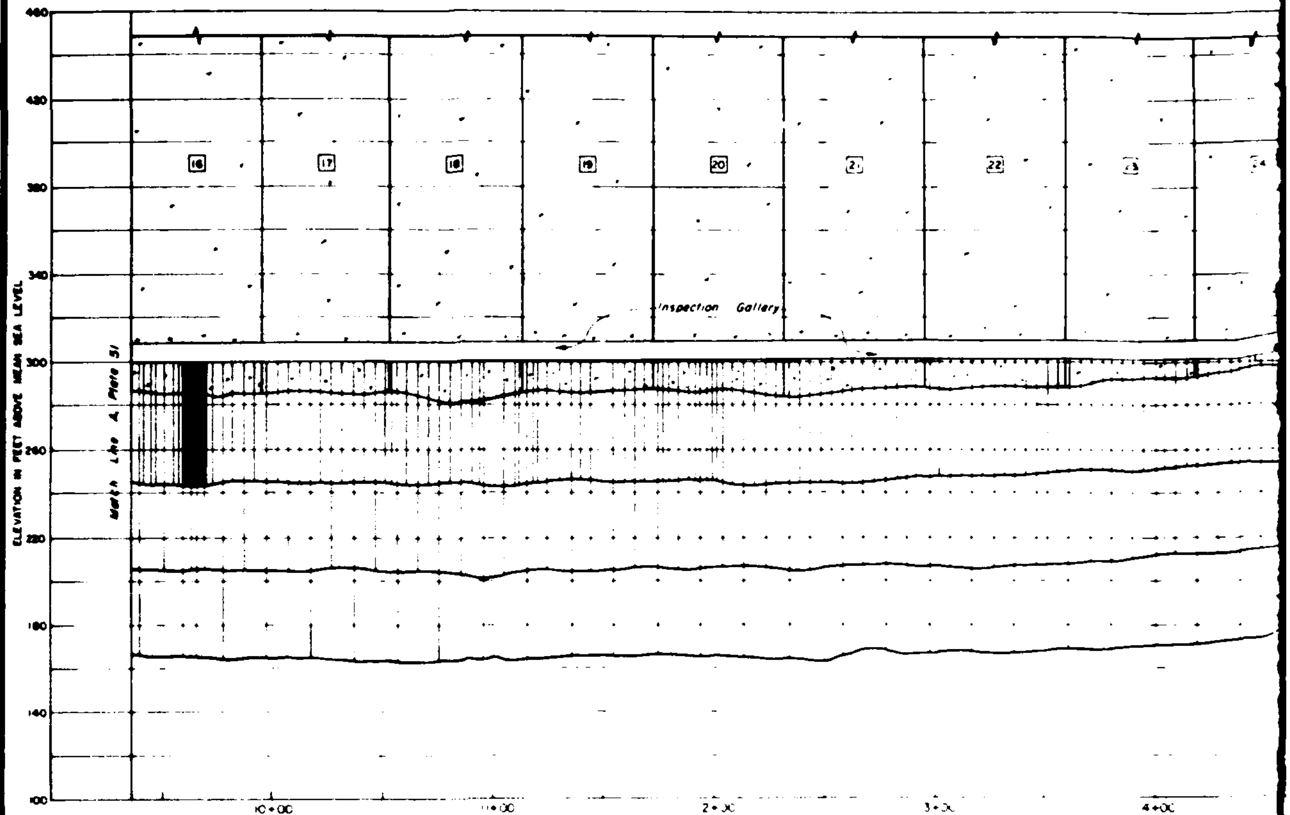


GROUT TAKES

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

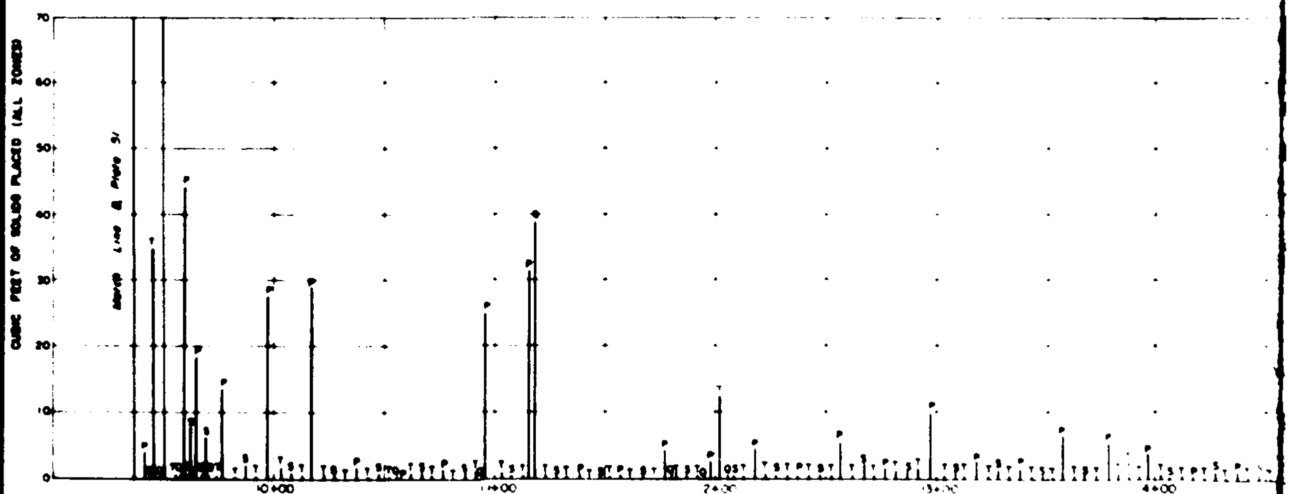
FOUNDATION REPORT
CONCRETE DAM
CURTAIN GROUTING PROFILE
AND GROUT TAKES (SHEET 1 OF 2)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 51



CURTAIN GROUT

SCALE



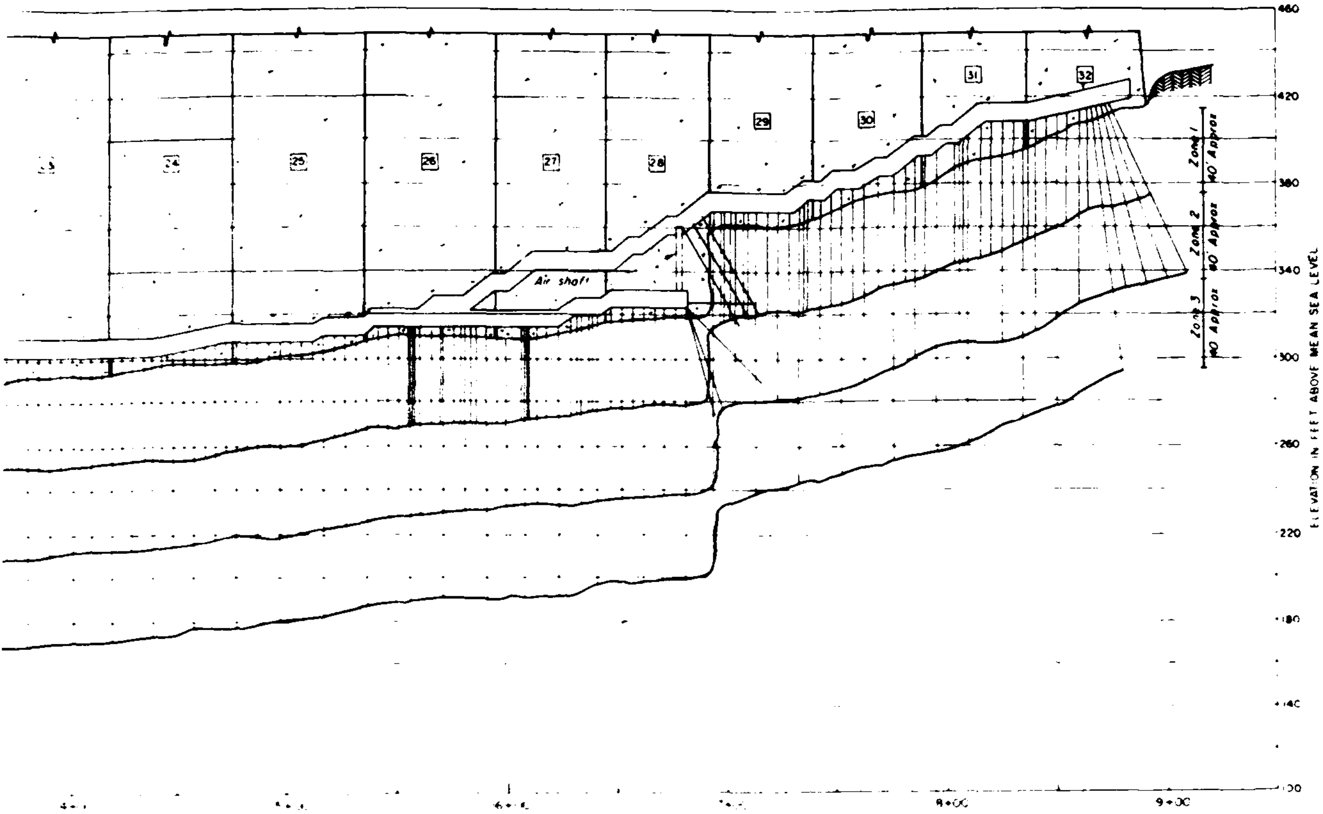
GROUT T

LEGEND

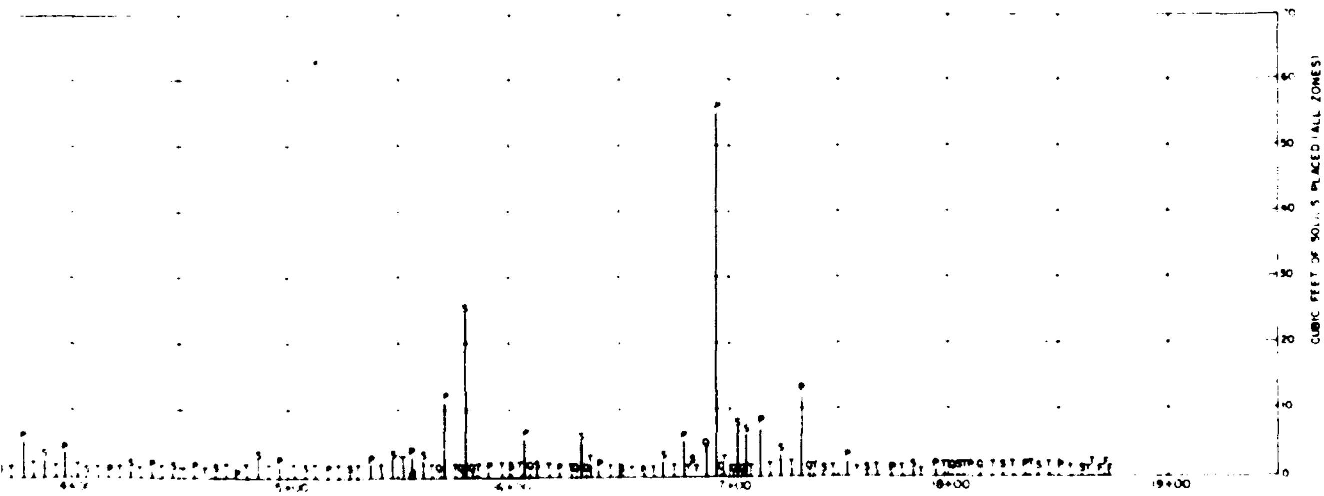
- Primary grout hole
- Secondary grout hole
- Tertiary grout hole
- Quaternary grout hole
- Fan grout hole

NOTE

See Plate 51 for notes



CURTAIN GROUTING PROFILE



GROUT TAKES

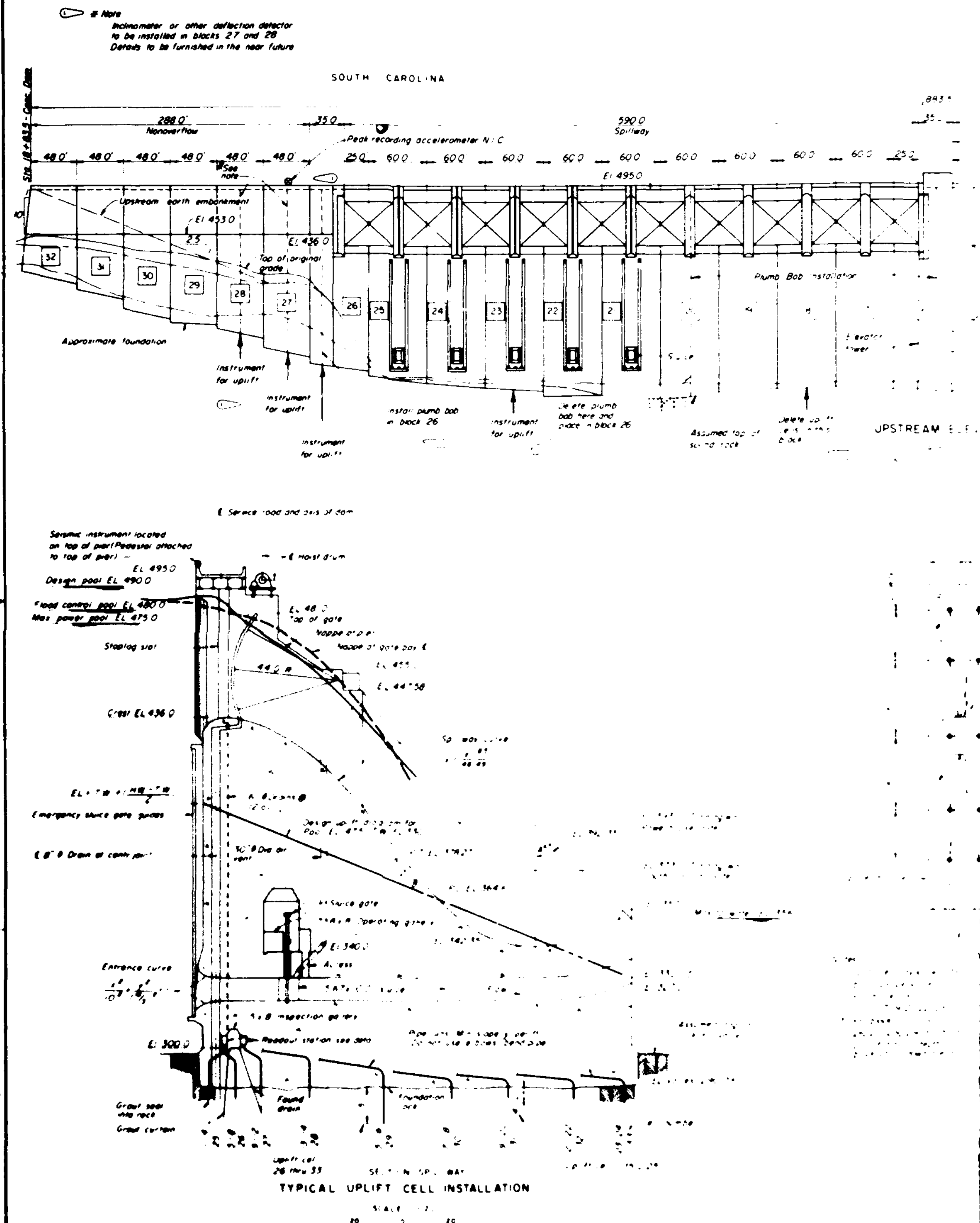
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
CURTAIN GROUTING PROFILE
AND GROUT TAKES (SHEET 2 OF 2)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 52

REVISIONS

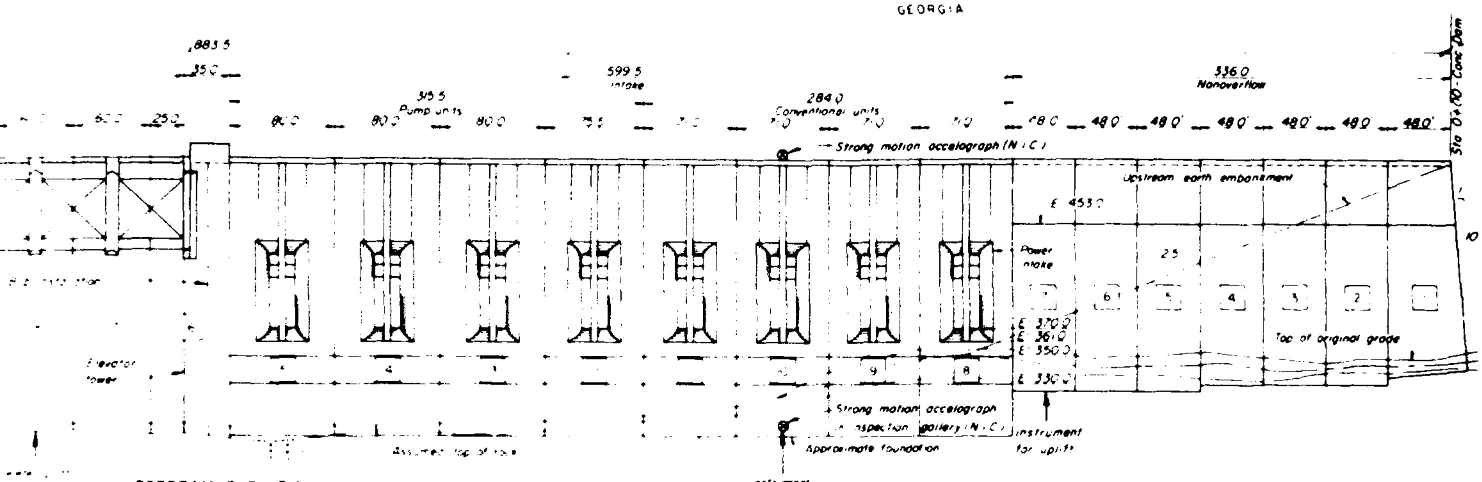
CORPS OF ENGINEERS



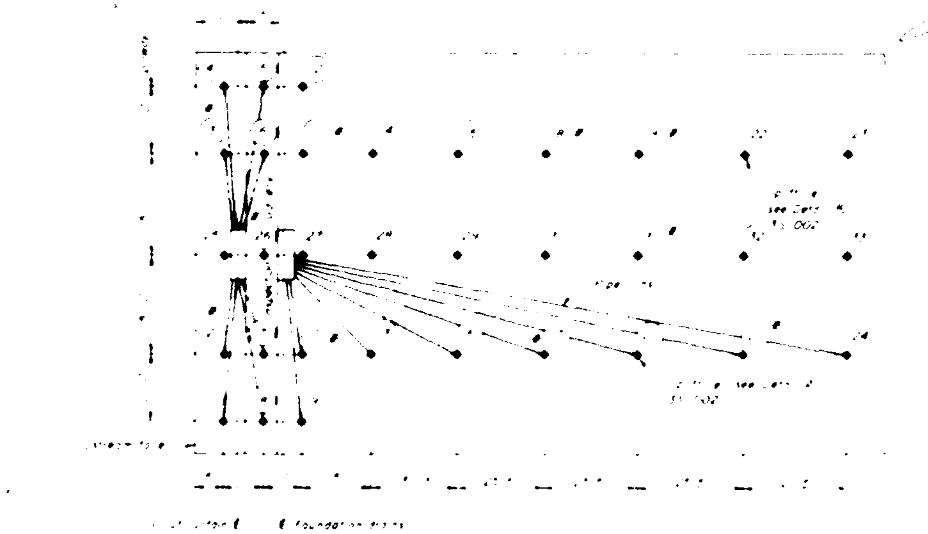
SECTION SPILLWAY
TYPICAL UPLIFT CELL INSTALLATION

U. S. ARMY

REVISIONS				
SYMBOL	ZONE	DESCRIPTION	DATE	BY
1	-	REVISED IN ACCORDANCE WITH MCB NO. 78-84-1	8 FEB 79	



UPSTREAM ELEVATION



LAYOUT OF UPLIFT CELLS

NOT TO SCALE

NOTES

1. ½" P.V.C. Tubing used to backflush cell to insure unobstructed pipe. P.V.C. tubing to be rated for 200 psi.
2. Blocks 7, 8, 23 and 28 are shown tentatively for uplift instrumentation. A minimum of four (4) blocks is necessary to get a sufficient sample of the hydrostatic pressure conditions under the dam.
3. Final determination of blocks to be instrumented will be made after excavation.
4. The vertical run of pipe for each cell will be kept in true vertical alignment. Pipes shall be placed to avoid air traps. Piping shall be braced during concrete placement to maintain alignment.
5. All pipes will be kept clear of debris. Plugs will be used during construction to prevent concrete or other matter from plugging pipes.
6. The bottom elevation of the vertical run of pipe and the foundation elevation of each cell shall be recorded at the time of installation.
7. Removable test gauges ... contract.
8. For general notes see ...
9. Seismic instrument blockout to have hinged sheet metal doors similar to uplift cell reading station with lighting and power supply.
10. Instrumentation materials shown on this drawing shall be furnished and installed under payment item 140.

REFERENCE DRAWINGS

RR...	Plan Of Excavation For Concrete Dam Monoliths 1 Thru 6 And Powerhouse
RRSC ...2	Plan And Sections Of Excavation For Concrete Dam Monoliths 7 Thru 32
RRS...	Galleries Plans And Elevation
RRS ...9	Outline And Details Block 6 Sheet 1 Of 2
RRS 2...	Outline And Details Block 6 Sheet 2 Of 2
RRS 2...	Outline And Details Block 7 Thru 20
RRS 2...	Outline And Details Block 2... Thru 25 Sheet 1 Of 4
RRS 2...	Outline And Details Piers 7 Thru 25 Sheet 1 Of 2
RRS 30...	Outline And Details Blocks 8 Thru 15 Sheet 1 Of 5

COMPANION DRAWINGS

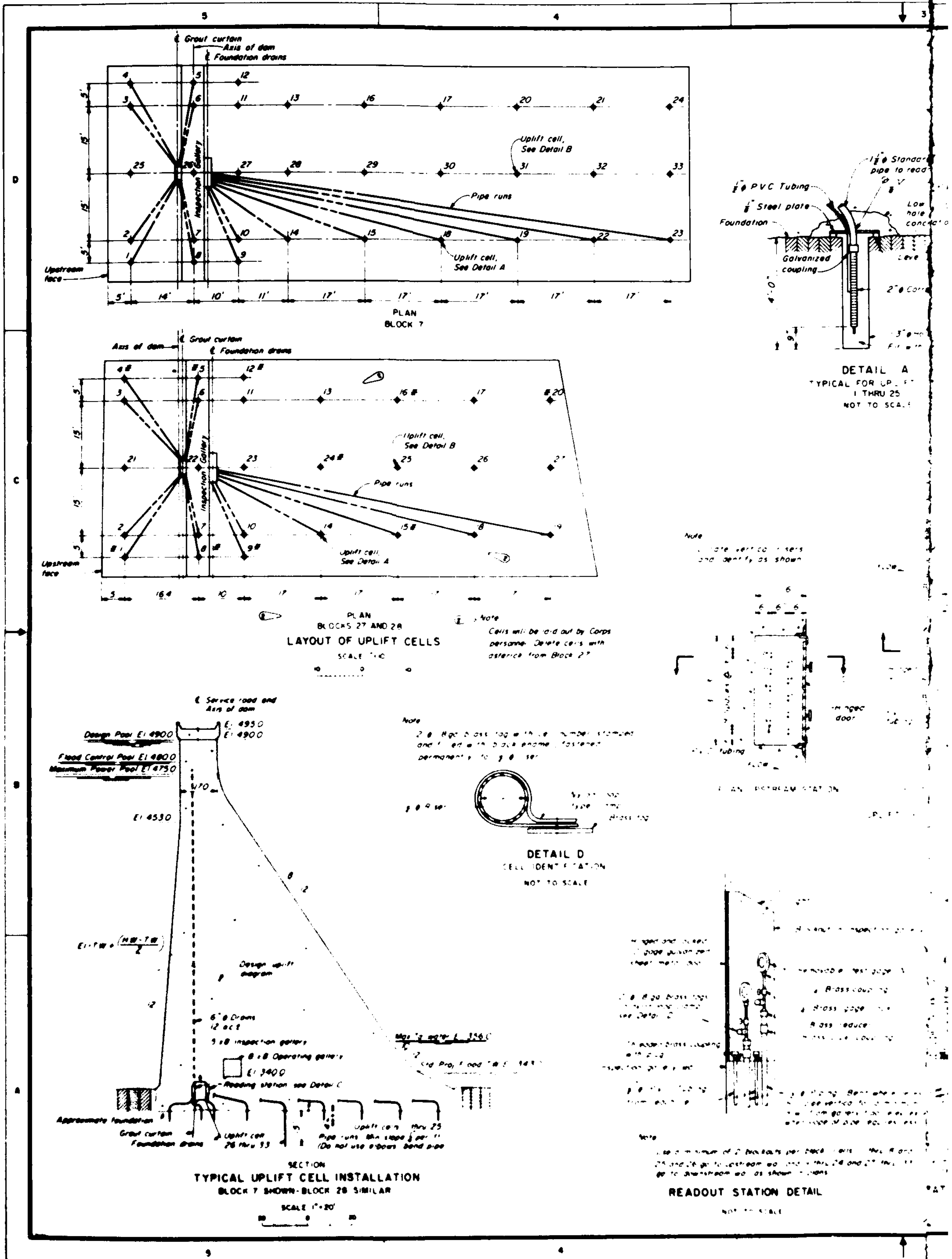
- IS-002
- IS-003
- IS-004
- IS-005
- IS-006

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS

FOUNDATION REPORT
CONCRETE DAM

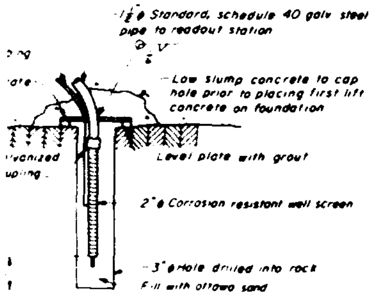
INSTRUMENTATION SHEET 1 OF 5
UPLIFT CELL & SEISMIC INSTRUMENT LOCATION
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 53

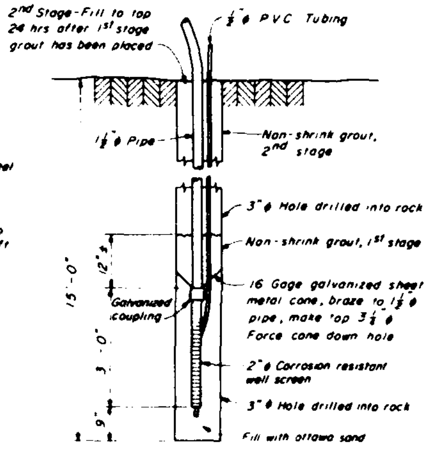


℄ Grout curtain
Axis of dam
℄ Foundation drains
Uplift cell, See Detail B
Pipe runs
Uplift cell, See Detail A
Upstream face
Inspection Gallery
PLAN
BLOCK 7
PLAN
BLOCKS 27 AND 28
LAYOUT OF UPLIFT CELLS
Cells will be laid out by Corps personnel. Delete cells with asterisk from Block 27
℄ Service road and Axis of dam
Design Pool El 4900
Flood Control Pool El 4800
Maximum Power Pool El 4750
El 4950
El 4900
El 4530
Design uplift diagram
6" ø Drains 12' o.c.
5 x 8 inspection gallery
8 x 8 Operating gallery
El 3400
Reading station see Detail C
Approximate foundation
Grout curtain
Foundation drains
Uplift cell 26 thru 33
Uplift cells 1 thru 25
Pipe runs. Min slope
(Do not use elbows. Bend pipe)
SECTION
TYPICAL UPLIFT CELL INSTALLATION
BLOCK 7 SHOWN-BLOCK 28 SIMILAR
SCALE 1"=20'
DETAIL D
NOT TO SCALE
PVC Tubing
Steel plate
Foundation
Galvanized coupling
DETAIL A
TYPICAL FOR UPLIFT 1 THRU 25
NOT TO SCALE
READOUT STATION DETAIL

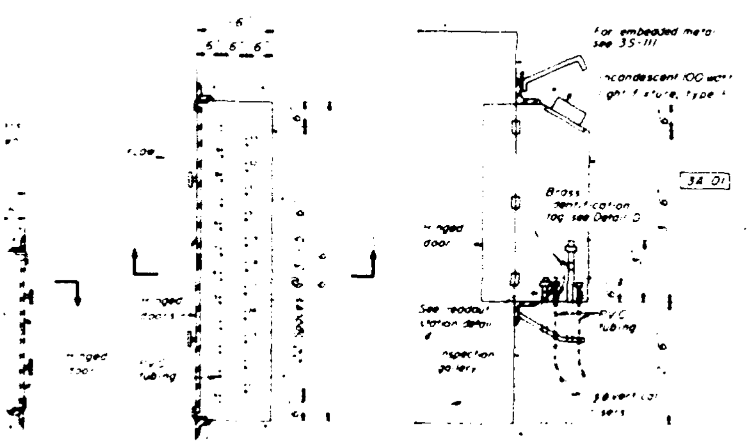
REVISIONS				
SYMBOL	ZONE	DESCRIPTION	DATE	BY
1	3C-28-18	REVISED IN ACCORDANCE WITH AMENDMENT NO 0008	25 APR 78	
2	-	REVISED IN ACCORDANCE WITH MCB NO. 78-64-1	9 FEB 79	



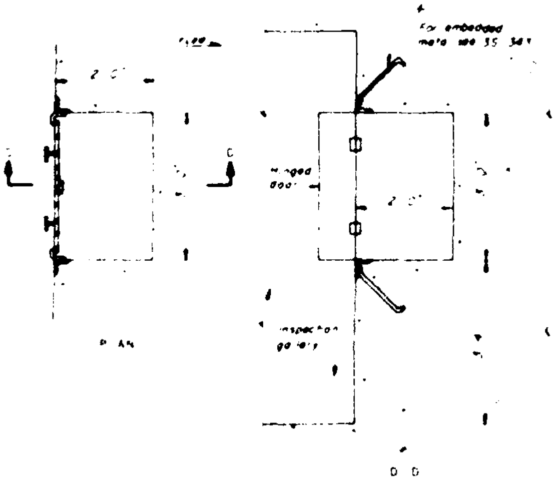
DETAIL A
TYPICAL FOR UPLIFT CELLS
1 THRU 25
NOT TO SCALE



DETAIL B
TYPICAL FOR UPLIFT CELLS
26 THRU 33
NOT TO SCALE



PLAN DOWNSTREAM STATION
DETAIL C
UPLIFT CELL TERMINAL READING STATION
SCALE ½" = 1'-0"



DETAIL E
STRONG MOTION ACCELEROGRAPH RECESS
SCALE ½" = 1'-0"

ETAIL

NOTE

NOTES

1. Materials shown on this drawing shall be furnished and installed under payment item 13A 01.
2. For additional reference drawings see 35-001
3. All cells shall be checked to see if they are open prior to each concrete lift placement This shall be done by pumping water thru PVC tubing and checking flow from ½" ø pipe Failure to keep cells open shall void payment for each cell not found open
4. Elevation of bottom of hole and top of foundation shall be recorded for each cell
5. When grouting all cells shall be pressurized with 5 PSI water pressure Do not use air

REFERENCE DRAWINGS

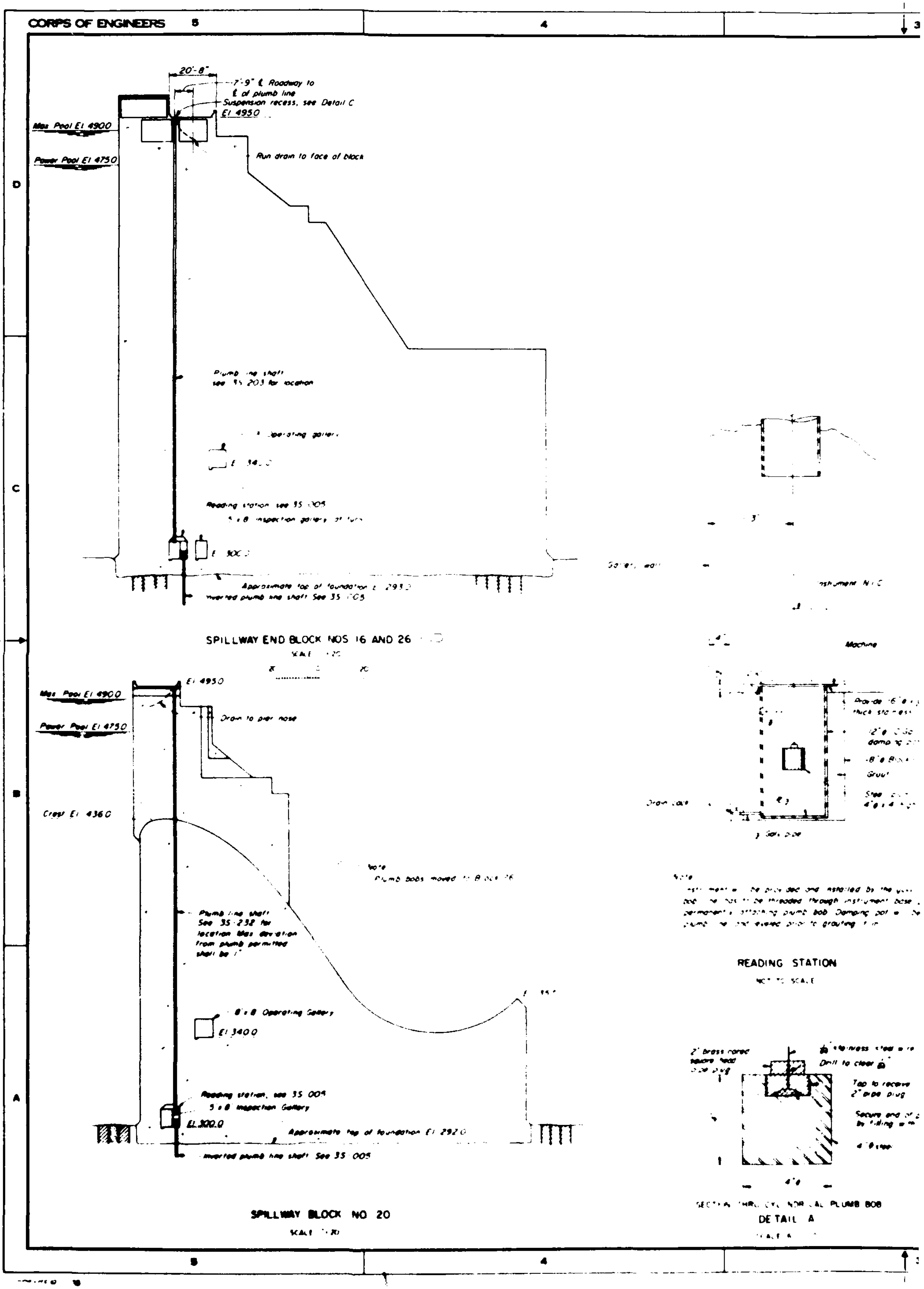
RR35 Miscellaneous Iron And Steel
RR35 343 Miscellaneous Metal

COMPANION DRAWINGS

35-001
35-003
35-004
35-005
35-006

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
CONCRETE DAM
INSTRUMENTATION SHEET 2 OF 5
UPLIFT CELLS
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
54

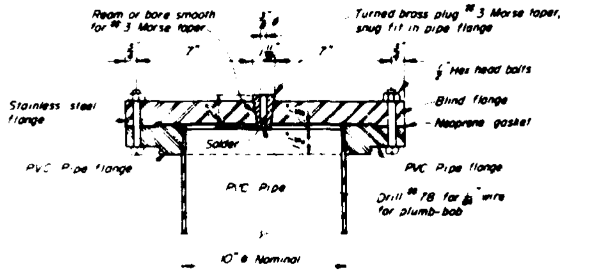
54



CORPS OF ENGINEERS
20'-8"
7'-9" ℄ Roadway to ℄ of plumb line
Suspension recess, see Detail C
El 4950
Max Pool El 4900
Power Pool El 4750
Run drain to face of block
Approximate top of foundation
Inverted plumb line shaft See 35-005
SPILLWAY END BLOCK NOS 16 AND 26
El 4950
Max Pool El 4900
Power Pool El 4750
Drain to pier nose
Crest El 4360
Note
Plumb bobs moved to Block 16
Plumb line shaft See 35-232 for location Max deviation from plumb permitted shall be 1"
8'x8' Operating Gallery
El 340.0
Reading station, see 35-005
5'x8' Inspection Gallery
El 300.0
Approximate top of foundation El 292.0
Inverted plumb line shaft See 35-005
SPILLWAY BLOCK NO 20
READING STATION
2" brass cored square head pipe plug
Drill to clear
Tap to receive 2" pipe plug
DETAIL A

U. S. ARMY

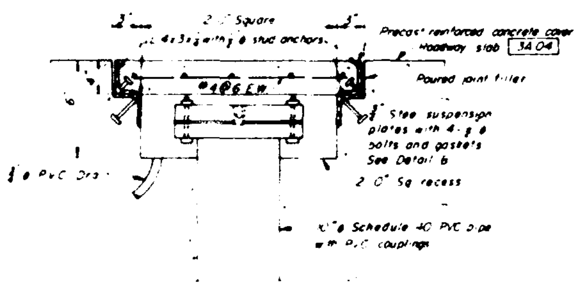
REVISIONS				
SYMBOL	ZONE	DESCRIPTION	DATE	BY
	-	REVISED IN ACCORDANCE WITH MCB NO 78-84-1	8 FEB 78	



SECTION THRU SUSPENSION PLATE

DETAIL B

SCALE 3" = 1'-0"

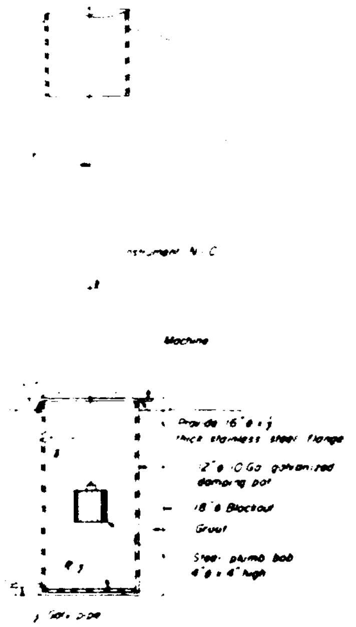


SUSPENSION RECESS

DETAIL C

NOTES:

1. Microscope - Base plate and reference plates provided by government.
2. Provide 2 plumb bob deflection facilities. Provide a reading station for each of the operating and inspection galleries.
3. Installation shall be complete prior to second stage diversion.
4. Materials shown on this drawing unless otherwise noted or specified shall be furnished and installed under payment item 13A01.
5. For reference drawings see 35-001.



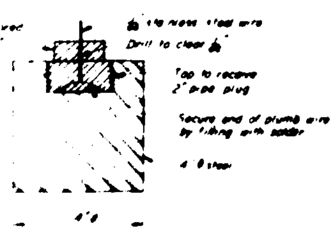
be provided and installed by the government. Plumb ... be threaded through instrument base plate prior to attaching plumb bob. Damping pot will be centered on and leveled prior to grouting it in.

READING STATION

NOT TO SCALE

COMPANION DRAWINGS:

35-001
35-002
35-003
35-005
35-006



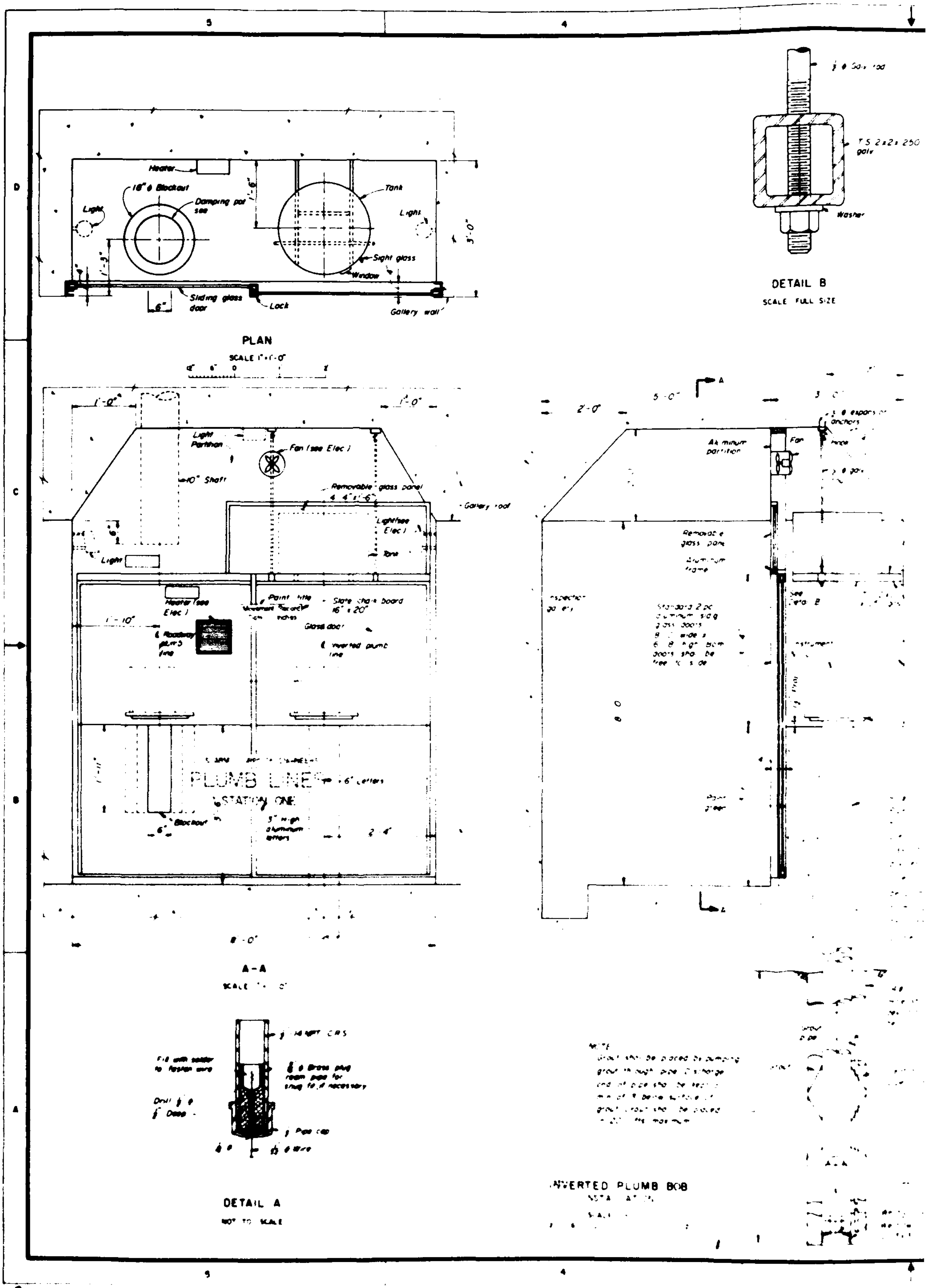
NORMAL PLUMB BOB

DETAIL A

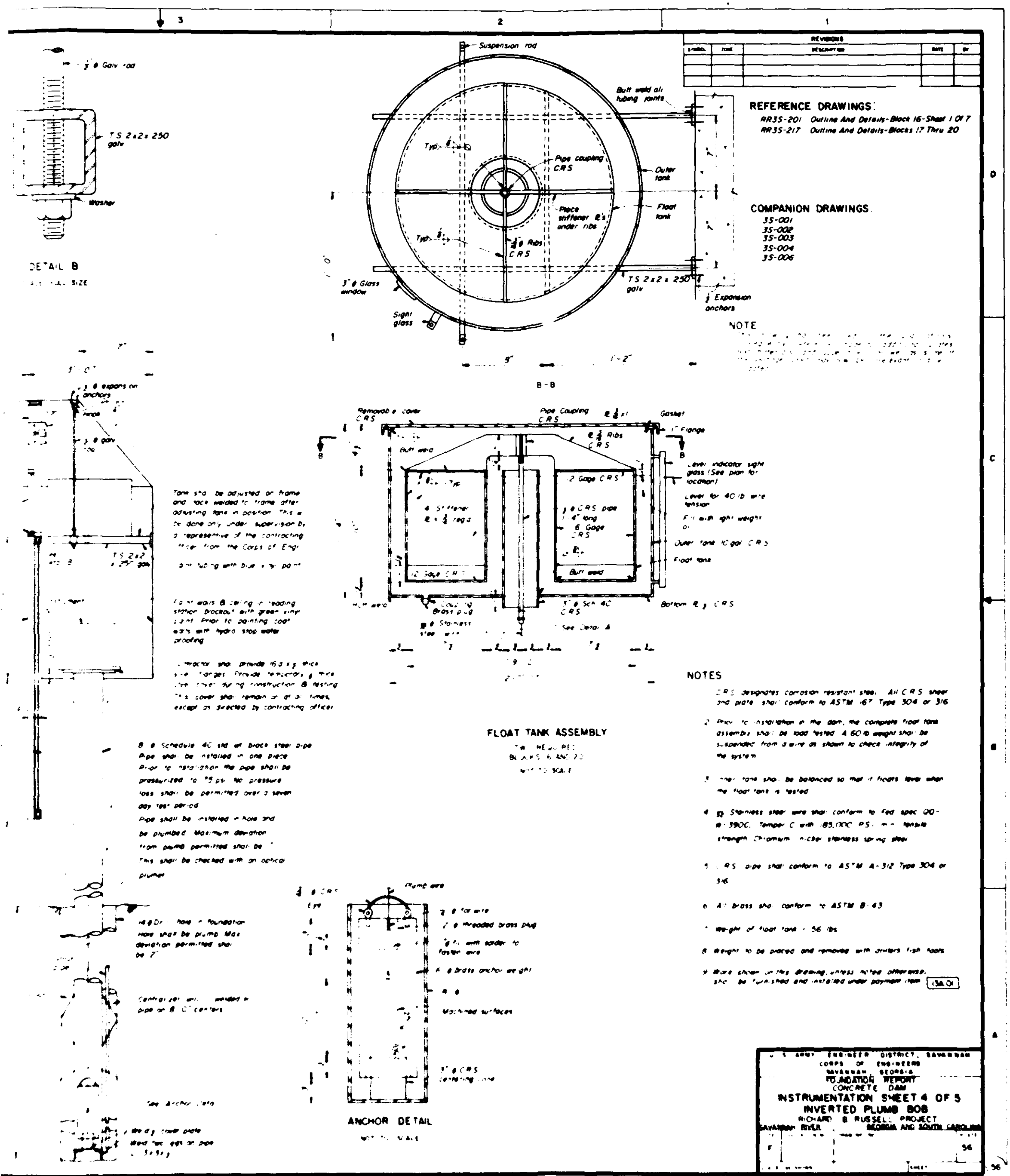
U S ARMY ENGINEER DISTRICT SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
INSTRUMENTATION SHEET 3 OF 5
PLUMB BOB
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

F | 35

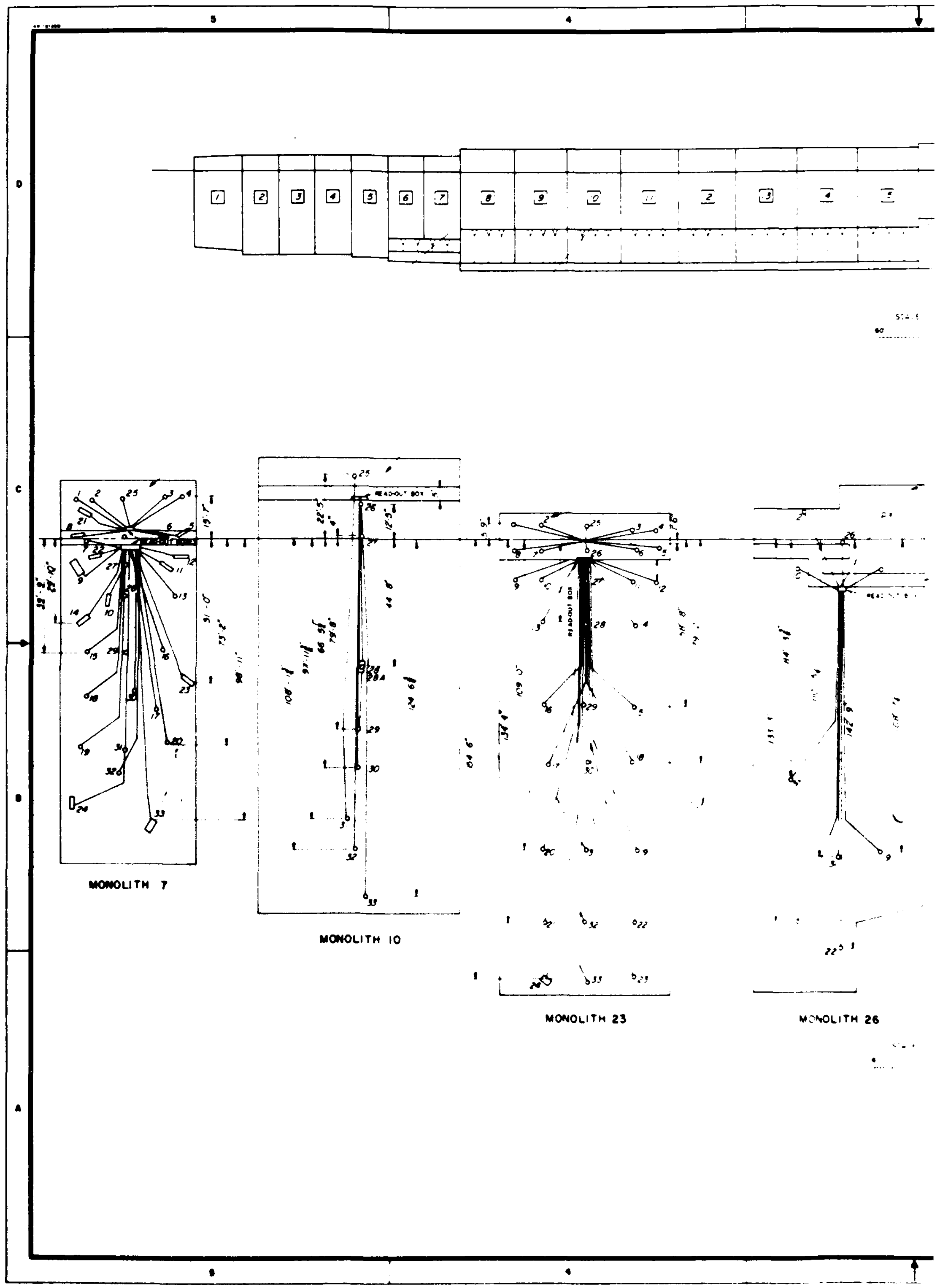


Heater
18" ø Blockout
Damping pot see
Tank
Light
Light
Sight glass
Window
Sliding glass door
Lock
Gallery wall
6"
3'-0"
PLAN
SCALE 1"=1'-0"
DETAIL B
SCALE FULL SIZE
Washer
T S 2x2x 250 galv
Light Partition
Fan (see Elec)
10" Shaft
Removable glass panel
Gallery roof
Light(see Elec)
Tank
Light
Heater (see Elec)
Point title
Slate chalk board 16" x 20"
Glass door
℄ Inverted plumb line
℄ Roadway plumb line
PLUMB LINES
STATION ONE
6" Letters
3" High aluminum letters
Blockout
2'-4"
8'-0"
A-A
2'-0"
5'-0"
Aluminum partition
Fan
Removable glass pane
Aluminum frame
Inspection gallery
see Detail B
DETAIL A
NOT TO SCALE
Fill with solder to fasten wire
Brass plug ream pipe for snug fit if necessary
Pipe cap
Wire
INVERTED PLUMB BOB

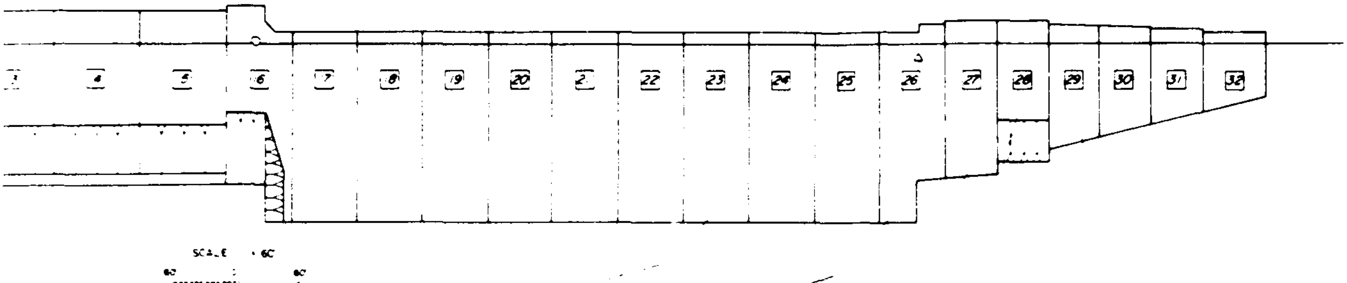


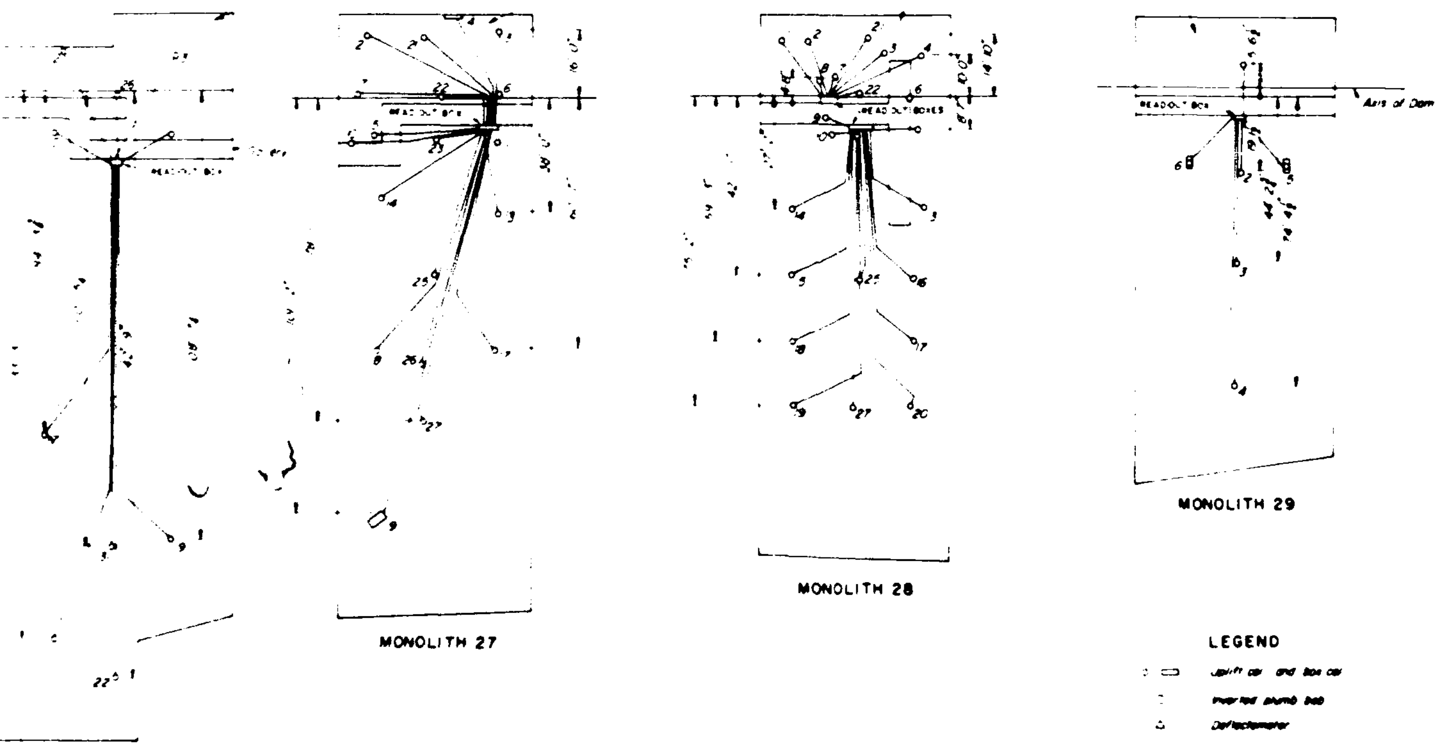
REVISIONS
SYMBOL
ZONE
DESCRIPTION
DATE
BY
REFERENCE DRAWINGS:
RR35-201 Outline And Details-Block 16-Sheet 1 Of 7
RR35-217 Outline And Details-Blocks 17 Thru 20
COMPANION DRAWINGS
35-001
35-002
35-003
35-004
35-006
NOTE
Suspension rod
Butt weld all tubing joints
Pipe coupling C R S
Outer tank
Float tank
Place stiffener R's under ribs
Typ
Ribs C R S
3" ø Glass window
Sight glass
T S 2 x 2 x 250 galv
Expansion anchors
B - B
DETAIL B
Galv rod
T S 2x2x 250 galv
Washer
Removable cover C R S
Pipe Coupling C R S
Gasket
Flange
Butt weld
Ribs C R S
Level indicator sight glass (See plan for location)
Lever for 40 lb wire tension
Fill with light weight oil
Outer tank 10 ga C R S
Float tank
12 Gage C R S
6 Gage C R S
Stiffener
Butt weld
Bottom C R S
Brass plug
Stainless steel wire
See Detail A
FLOAT TANK ASSEMBLY
NOT TO SCALE
Tank shall be adjusted on frame and tack welded to frame after adjusting tank in position. This is to be done only under supervision by a representative of the contracting officer from the Corps of Engr
Paint tubing with blue vinyl paint
Paint walls & ceiling in reading station blockout with green vinyl paint. Prior to painting coat walls with hydro stop water proofing
Contractor shall provide flanges. Provide temporary cover during construction & testing. This cover shall remain on at all times except as directed by contracting officer
8" ø Schedule 40 std wt black steel pipe. Pipe shall be installed in one piece. Prior to installation the pipe shall be pressurized to 75 psi. No pressure loss shall be permitted over a seven day test period.
Pipe shall be installed in hole and be plumbed. Maximum deviation from plumb permitted shall be
This shall be checked with an optical plummet
Hole in foundation. Hole shall be plumb. Max deviation permitted shall be 2"
Centralizer will be welded to pipe on 8' 0" centers
See Anchor Detail
Weld cover plate
Plumb wire
Eye
for wire
Threaded brass plug
Fill with solder to fasten wire
Brass anchor weight
Machined surfaces
C R S centering cone
ANCHOR DETAIL
NOT TO SCALE
NOTES
1. C R S designates corrosion resistant steel. All C R S sheet and plate shall conform to ASTM 167 Type 304 or 316
2. Prior to installation in the dam, the complete float tank assembly shall be load tested. A 60 lb weight shall be suspended from a wire as shown to check integrity of the system
3. Inner tank shall be balanced so that it floats level when the float tank is tested
4. Stainless steel wire shall conform to Fed spec QQ-W-390C, Temper C with 185,000 P S I min tensile strength Chromium nickel stainless spring steel
5. C R S pipe shall conform to ASTM A-312 Type 304 or 316
6. All brass shall conform to ASTM B 43
7. Weight of float tank - 56 lbs
8. Weight to be placed and removed with drillers fish tools
9. Work shown on this drawing, unless noted otherwise, shall be furnished and installed under payment item 13A.01
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
CONCRETE DAM
INSTRUMENTATION SHEET 4 OF 5
INVERTED PLUMB BOB
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER
GEORGIA AND SOUTH CAROLINA
56

1



5
4
D
C
B
A
READOUT BOX
MONOLITH 7
MONOLITH 10
MONOLITH 23
MONOLITH 26





MONOLITH 26

MONOLITH 27

MONOLITH 28

MONOLITH 29

LEGEND

Uplift cell and box cell

Inverted plumb bob

Deflectometer

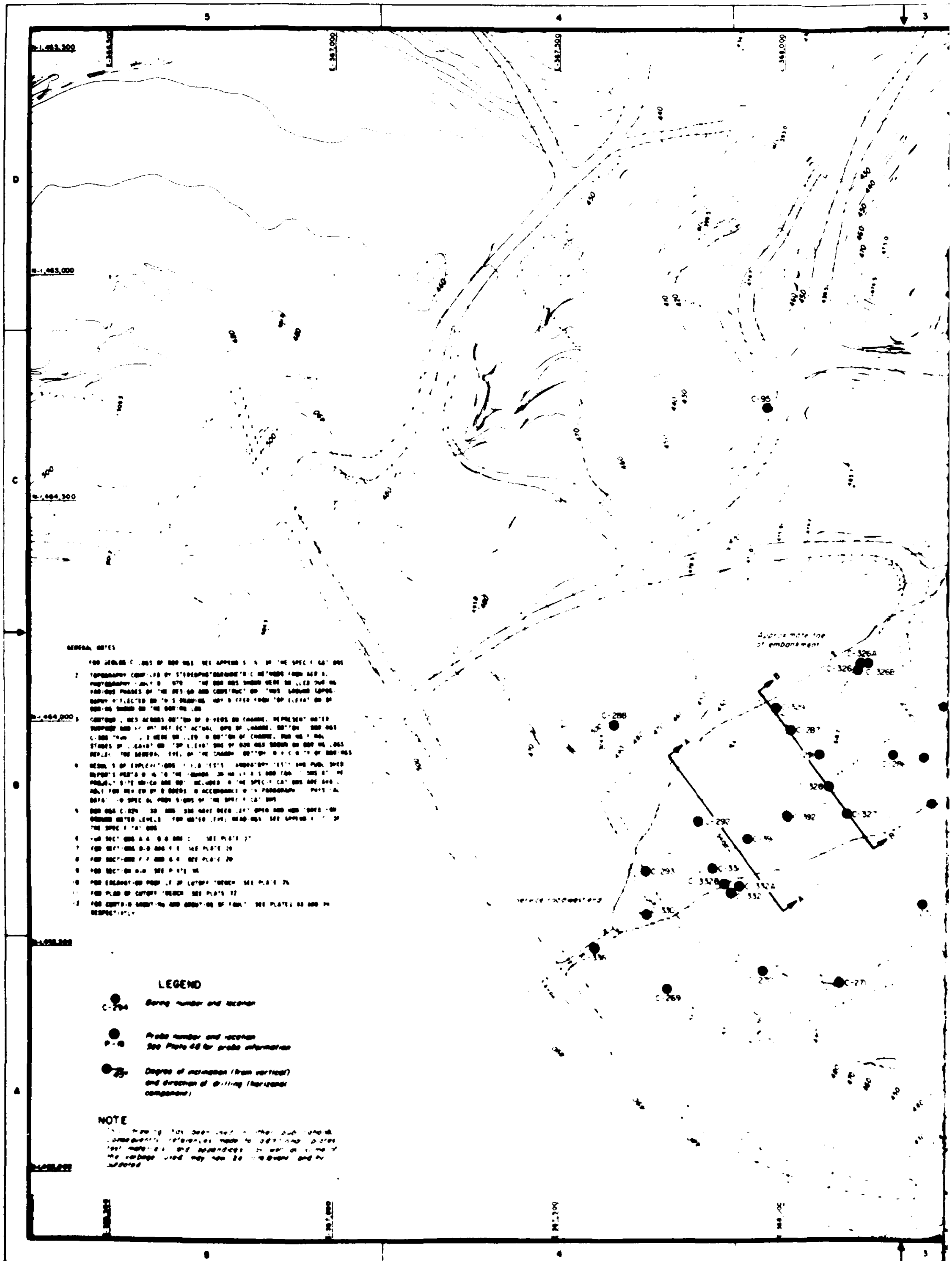
SYMBOL	ZONE	DESCRIPTION	DATE	BY
REVISIONS				

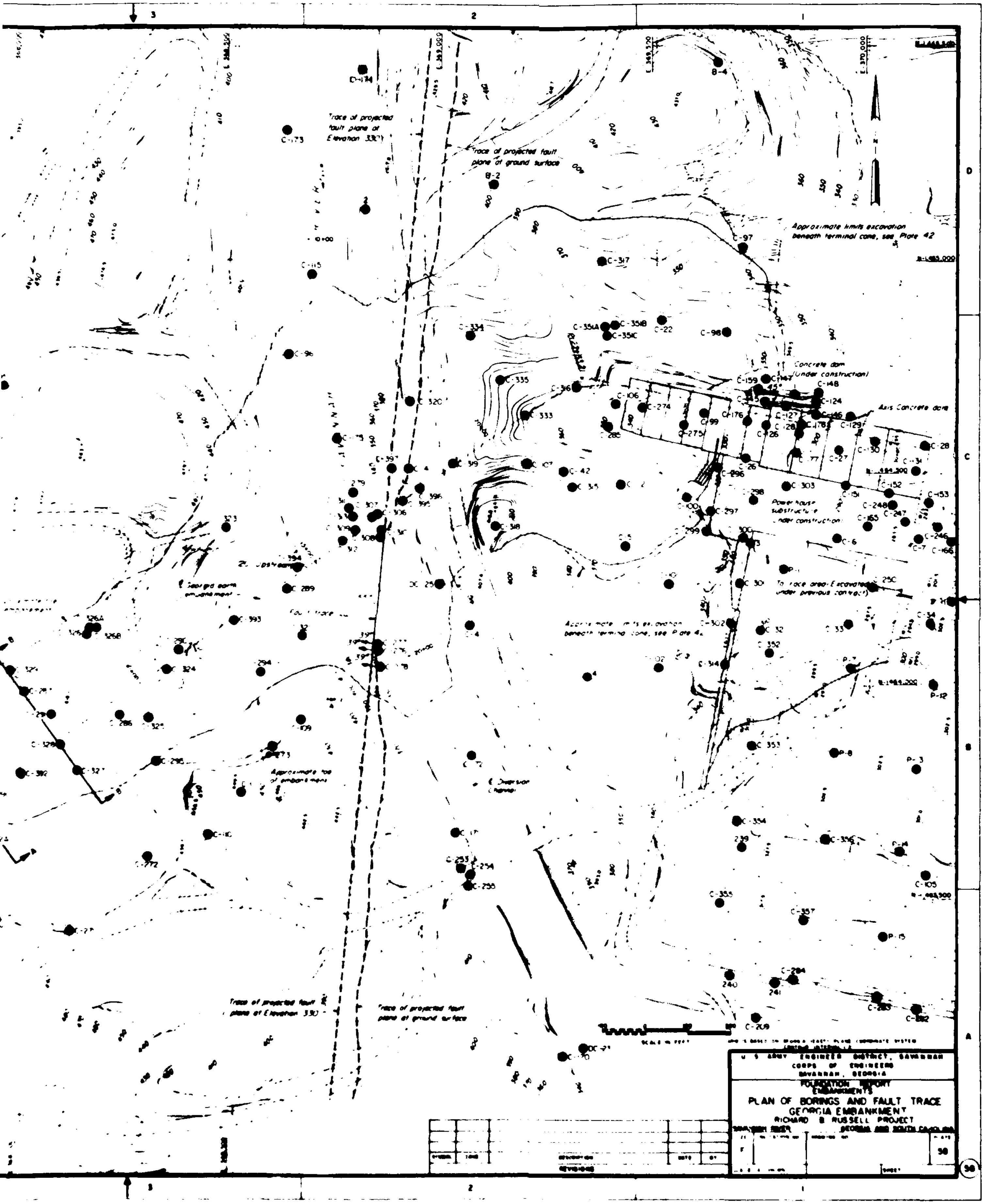
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
CONCRETE DAM
INSTRUMENTATION LOCATION
(AS-BUILT) SHEET 5 OF 5
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

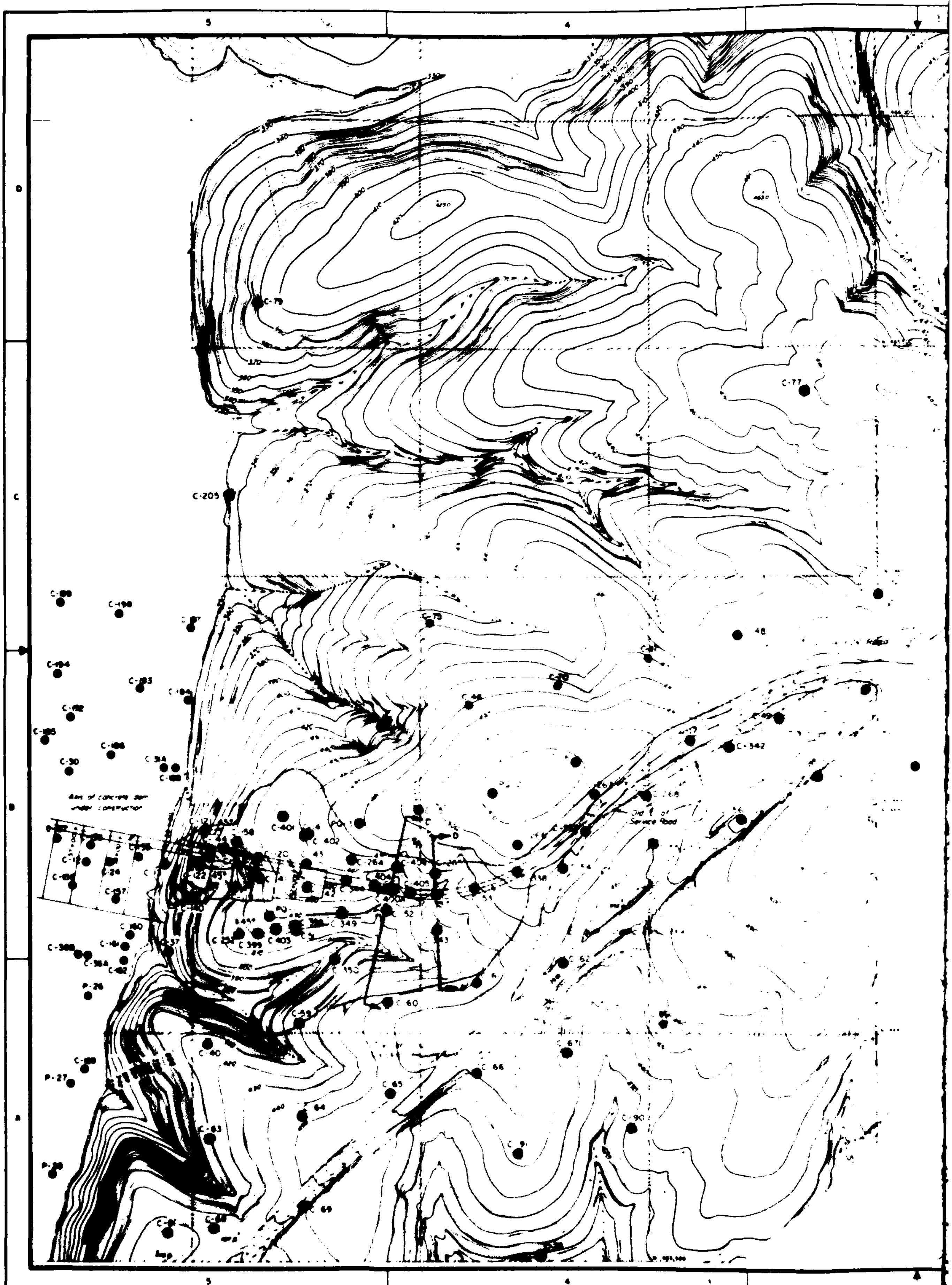
PLATE 57

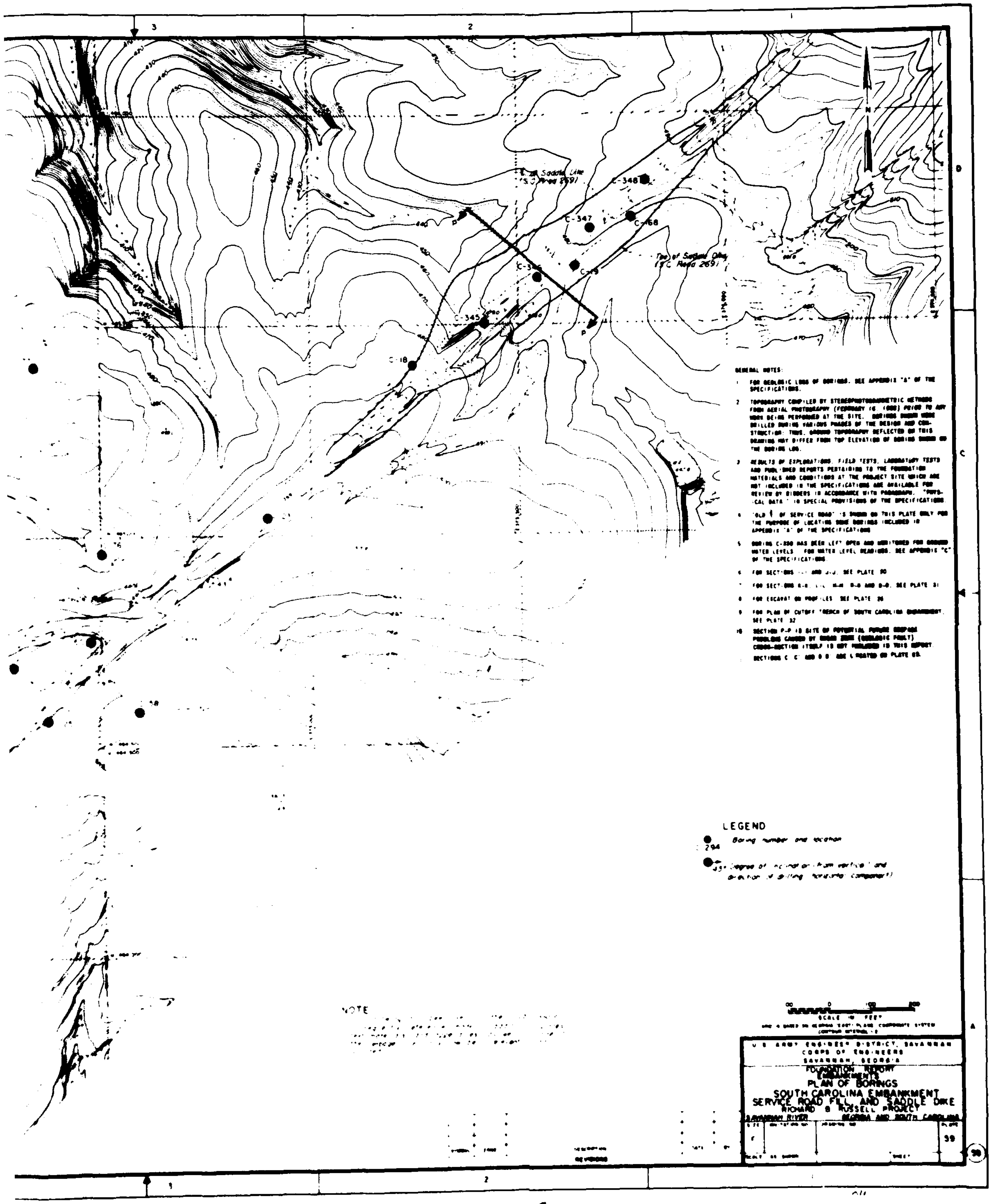
57


5
4
3
D
C
B
A
C-95
C-288
C-326A
C-326B
C-327
C-290
C-293
C-269
C-271
GENERAL NOTES
LEGEND
Boring number and location
Probe number and location
See Plate 46 for probe information
Degree of inclination (from vertical) and direction of drilling (horizontal component)
NOTE

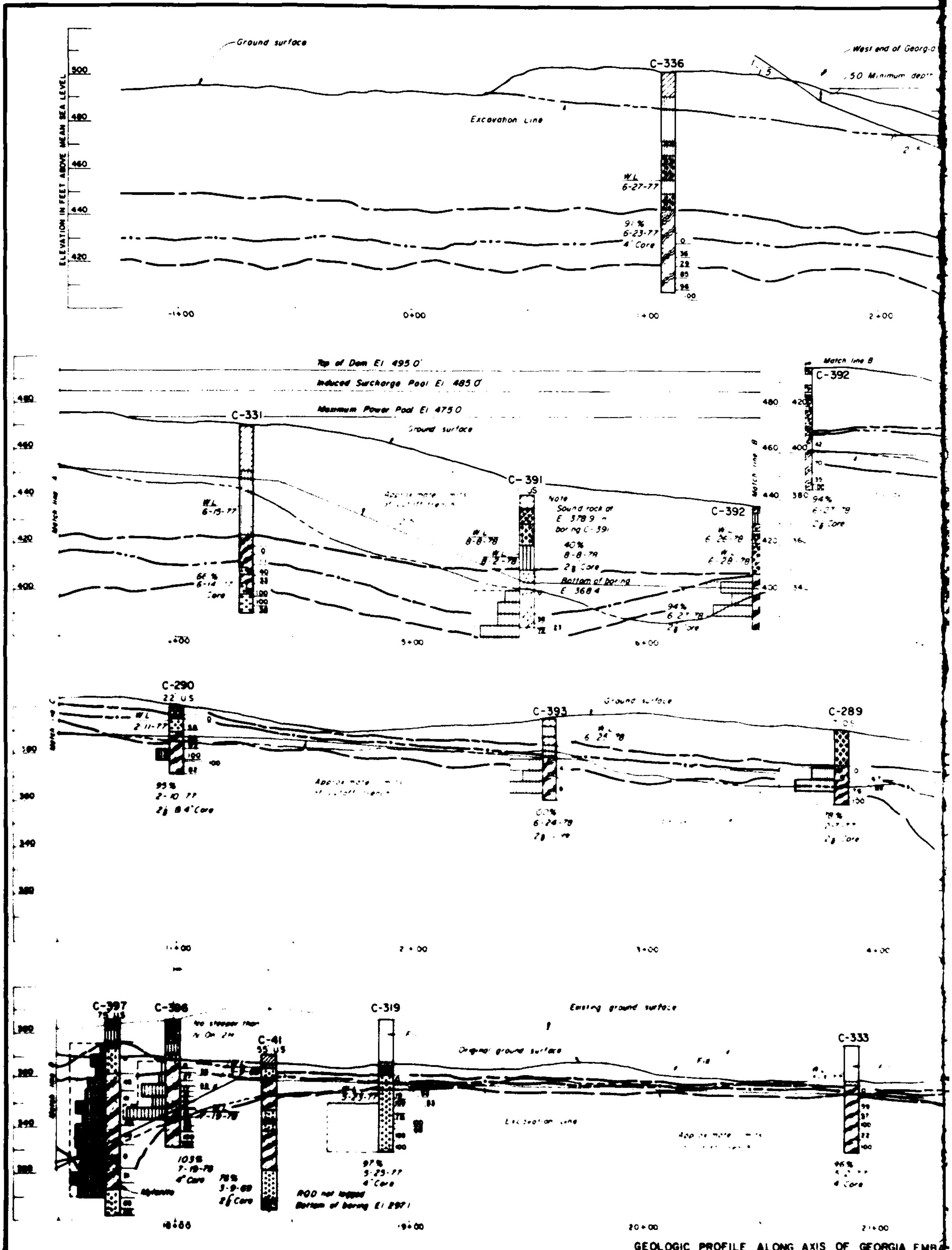


U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
PLAN OF BORINGS AND FAULT TRACE
GEORGIA EMBANKMENT
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER
GEORGIA AND SOUTH CAROLINA
38
Trace of projected fault plane at Elevation 330
Trace of projected fault plane at ground surface
Approximate limits excavation beneath terminal cone, see Plate 42
Concrete dam (Under construction)
Axis Concrete dam
Power house substructure under construction
Approximate toe of embankment
Diversion Channel
Fault trace
Georgia earth embankment

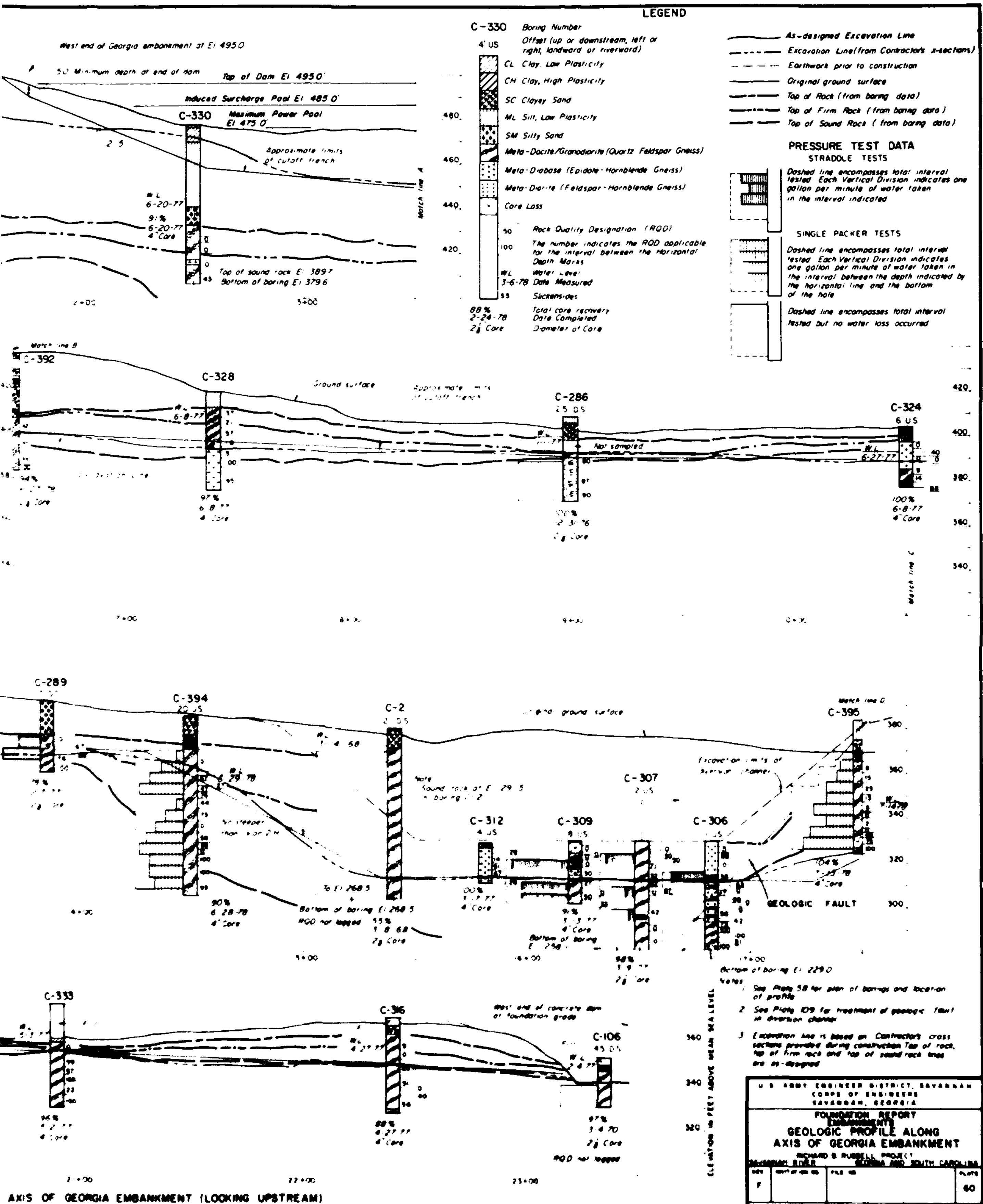

5
4
D
C
B
A
C-79
C-77
C-205
C-48
C-347
Old ℄ of Service Road
C-401
C-402
C-264
C-405
C-349
C-403
C-399
C-350
C-60
C-40
C-66
C-65
C-64
C-63
C-67
C-90
C-69
C-62
P-26
P-27
1


C-348
C-347
C-68
C-346
C-19
C-345
C-18
Top of Saddle Dike (S.C. Road 269)
GENERAL NOTES:
1 FOR GEOLOGIC LOGS OF BORINGS, SEE APPENDIX "A" OF THE SPECIFICATIONS.
2 TOPOGRAPHY COMPILED BY STEREOPHOTOGRAMMETRIC METHODS FROM AERIAL PHOTOGRAPHY PRIOR TO ANY WORK BEING PERFORMED AT THE SITE. BORINGS SHOWN WERE DRILLED DURING VARIOUS PHASES OF THE DESIGN AND CONSTRUCTION; THUS, GROUND TOPOGRAPHY REFLECTED ON THIS DRAWING MAY DIFFER FROM TOP ELEVATION OF BORING SHOWN ON THE BORING LOG.
3 RESULTS OF EXPLORATIONS, FIELD TESTS, LABORATORY TESTS AND PUBLISHED REPORTS PERTAINING TO THE FOUNDATION MATERIALS AND CONDITIONS AT THE PROJECT SITE WHICH ARE NOT INCLUDED IN THE SPECIFICATIONS ARE AVAILABLE FOR REVIEW BY BIDDERS IN ACCORDANCE WITH PARAGRAPH "PHYSICAL DATA" IN SPECIAL PROVISIONS OF THE SPECIFICATIONS.
4 "OLD ℄ OF SERVICE ROAD" IS SHOWN ON THIS PLATE ONLY FOR THE PURPOSE OF LOCATING SOME BORINGS INCLUDED IN APPENDIX "A" OF THE SPECIFICATIONS.
5 BORING C-330 HAS BEEN LEFT OPEN AND MONITORED FOR GROUND WATER LEVELS. FOR WATER LEVEL READINGS, SEE APPENDIX "C" OF THE SPECIFICATIONS.
6 FOR SECTIONS AND J-J, SEE PLATE 30
7 FOR SECTIONS AND D-D, SEE PLATE 31
8 FOR EXCAVATION PROFILES SEE PLATE 36
9 FOR PLAN OF CUTOFF TRENCH OF SOUTH CAROLINA EMBANKMENT, SEE PLATE 32
10 SECTION P-P IS SITE OF POTENTIAL FUTURE SEEPAGE PROBLEMS CAUSED BY SHEAR ZONE (GEOLOGIC FAULT). CROSS-SECTION ITSELF IS NOT INCLUDED IN THIS REPORT.
SECTIONS AND ARE LOCATED ON PLATE 33.
LEGEND
C-294 Boring number and location
Degree of inclination from vertical and direction of drilling (horizontal component)
NOTE
SCALE IN FEET
U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
PLAN OF BORINGS
SOUTH CAROLINA EMBANKMENT
SERVICE ROAD FILL AND SADDLE DIKE
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
PLATE 39
REVISIONS

2



GEOLOGIC PROFILE ALONG AXIS OF GEORGIA EMBA



Notes:
1. See Plate 58 for plan of borings and location of profile
2. See Plate 109 for treatment of geologic fault in diversion channel
3. Excavation line is based on Contractor's cross sections provided during construction. Top of rock, top of firm rock and top of sound rock lines are as-designed

U.S. ARMY ENGINEER DISTRICT, SAVANNAH
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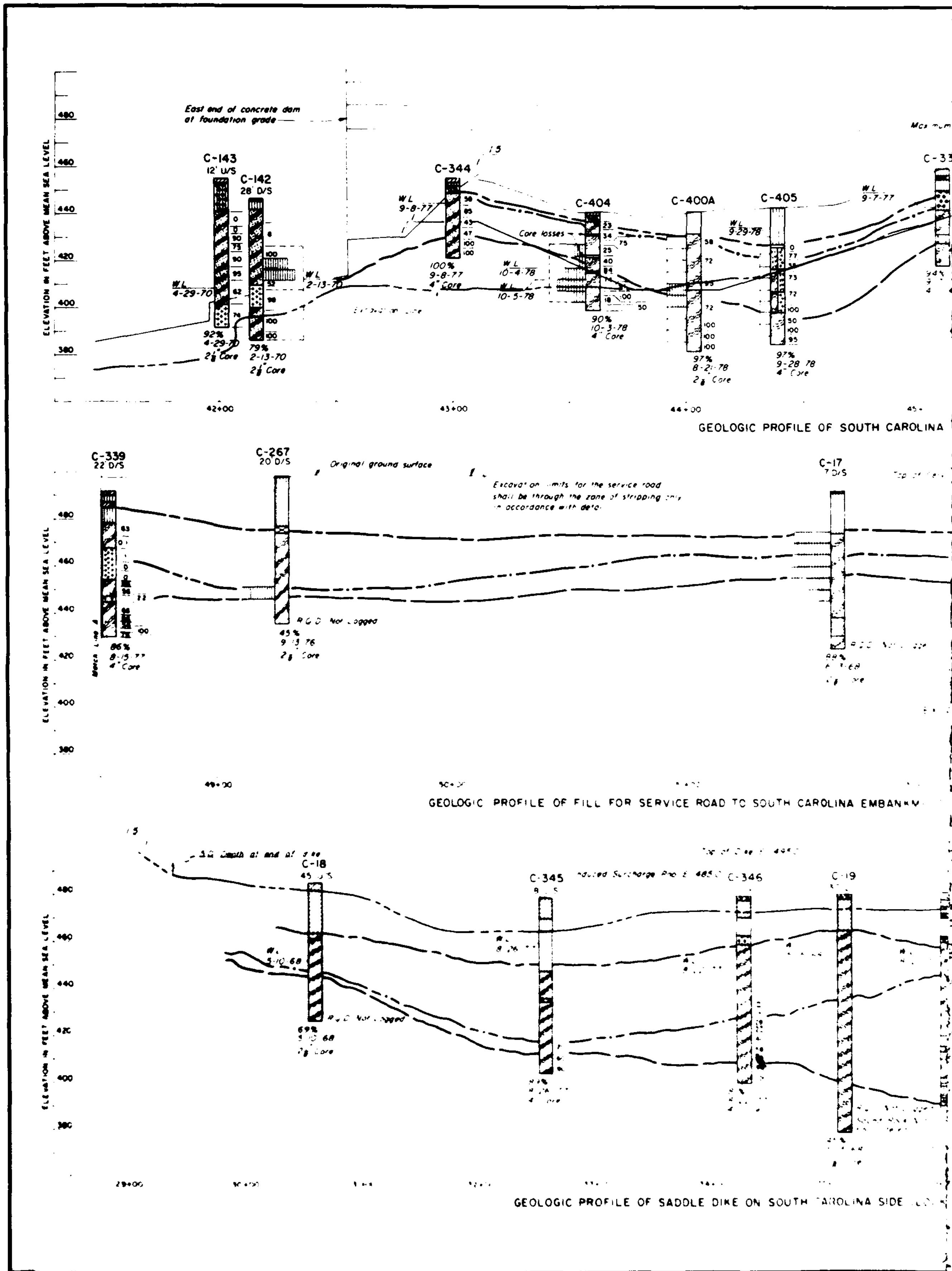
SAVANNAH, GEORGIA

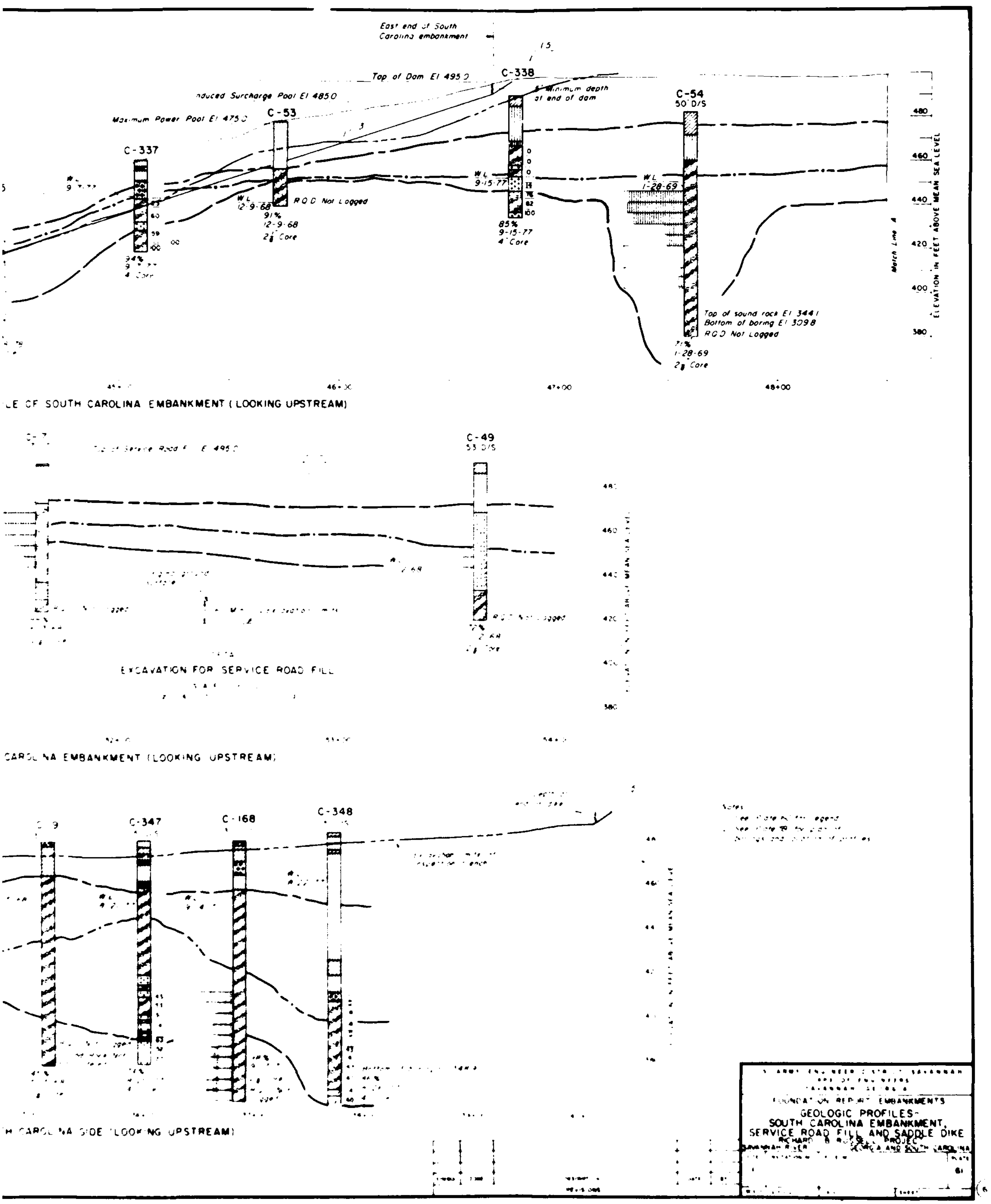
FOUNDATION REPORT
EMBANKMENTS
GEOLOGIC PROFILE ALONG
AXIS OF GEORGIA EMBANKMENT

RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

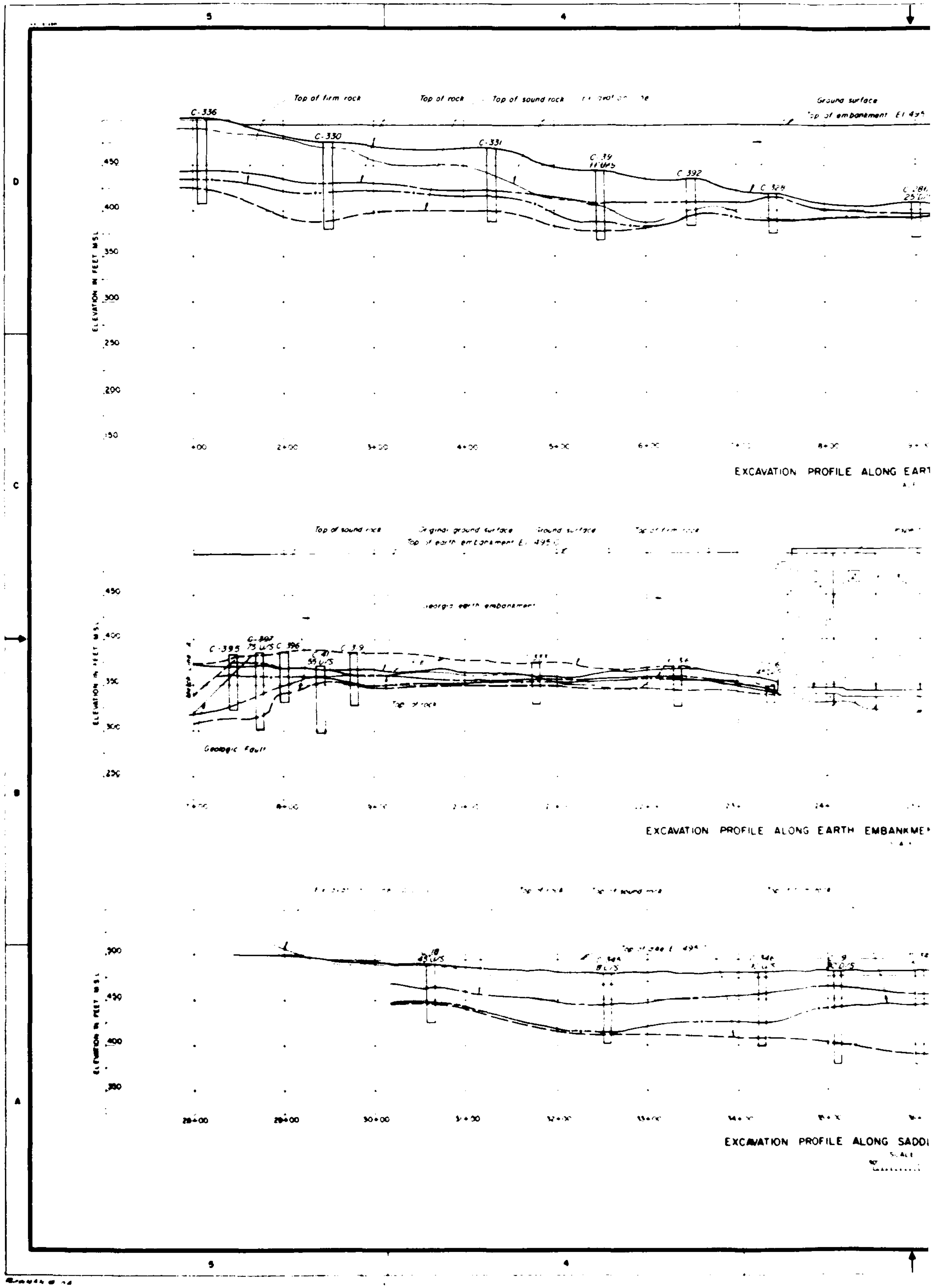
PLATE 60



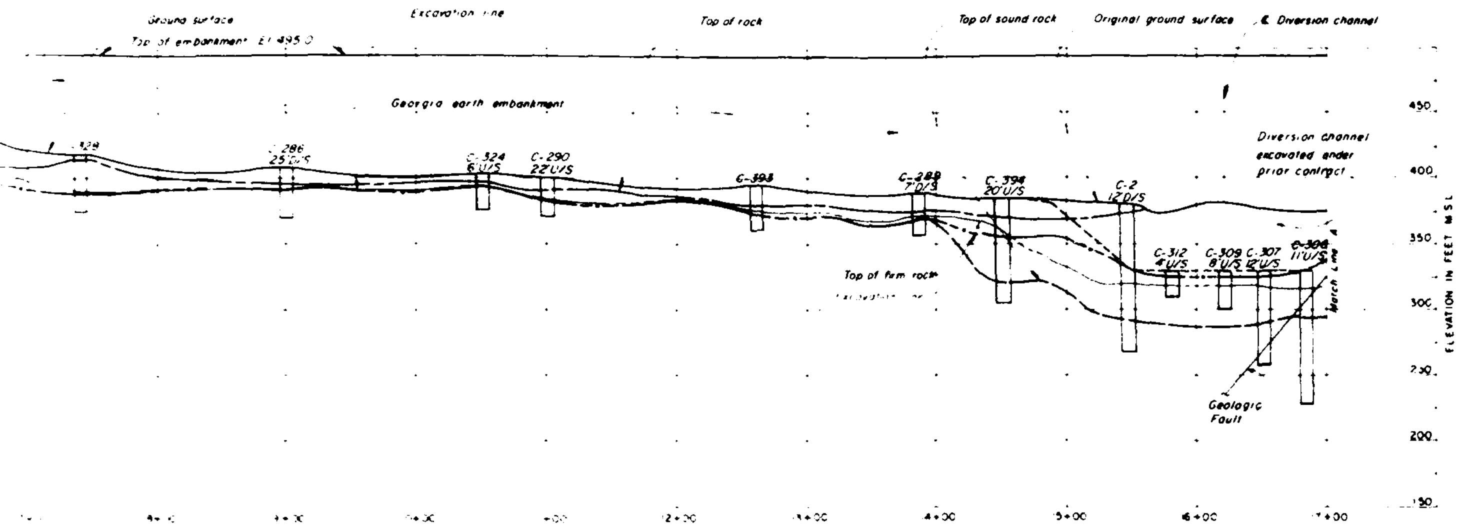
East end of concrete dam at foundation grade
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
C-143
12' U/S
C-142
28' D/S
C-344
C-404
C-400A
C-405
W.L. 9-8-77
W.L. 4-29-70
W.L. 2-13-70
W.L. 10-4-78
W.L. 10-5-78
W.L. 9-29-78
W.L. 9-7-77
Core losses
92% 4-29-70 2 1/8" Core
79% 2-13-70 2 1/8" Core
100% 9-8-77 4" Core
90% 10-3-78 4" Core
97% 8-21-78 2 1/8" Core
97% 9-28-78 4" Core
42+00
43+00
44+00
GEOLOGIC PROFILE OF SOUTH CAROLINA
C-339
22' D/S
C-267
20' D/S
C-17
Original ground surface
Excavation limits for the service road shall be through the zone of stripping only in accordance with detail
R.Q.D. Not Logged
86% 8-15-77 4" Core
45% 9-13-76 2 1/8" Core
49+00
GEOLOGIC PROFILE OF FILL FOR SERVICE ROAD TO SOUTH CAROLINA EMBANKMENT
C-18
45' U/S
C-345
C-346
C-19
R.Q.D. Not Logged
69% 5-10-68 2 1/8" Core
29+00
GEOLOGIC PROFILE OF SADDLE DIKE ON SOUTH CAROLINA SIDE



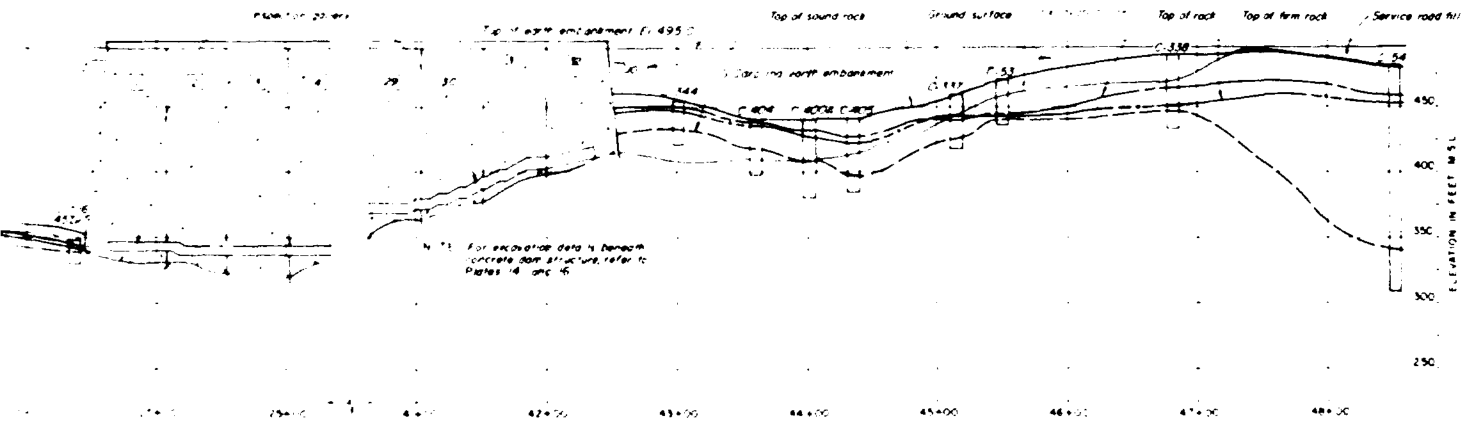
East end of South Carolina embankment
Top of Dam El 495.0
C-338
Induced Surcharge Pool El 485.0
5' Minimum depth at end of dam
C-54
50' D/S
Maximum Power Pool El 475.0
C-53
C-337
W.L. 9-15-77
W.L. 1-28-69
W.L. 12-9-68
R.Q.D. Not Logged
91%
12-9-68
2 1/8" Core
85%
9-15-77
4" Core
94%
4" Core
Top of sound rock El 344.1
Bottom of boring El 309.8
R.Q.D. Not Logged
71%
1-28-69
2 1/8" Core
Match Line A
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
480
460
440
420
400
380
46+00
47+00
48+00
...LE OF SOUTH CAROLINA EMBANKMENT (LOOKING UPSTREAM)
C-49
53' D/S
R.Q.D. Not Logged
EXCAVATION FOR SERVICE ROAD FILL
...CAROLINA EMBANKMENT (LOOKING UPSTREAM)
C-347
C-168
C-348
...H CAROLINA SIDE (LOOKING UPSTREAM)
FOUNDATION REPORT EMBANKMENTS
GEOLOGIC PROFILES-
SOUTH CAROLINA EMBANKMENT,
SERVICE ROAD FILL AND SADDLE DIKE
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA



Top of firm rock
Top of rock
Top of sound rock
Ground surface
C-336
C-330
C-331
C-392
ELEVATION IN FEET M.S.L.
EXCAVATION PROFILE ALONG EART
Top of sound rock
Original ground surface
Ground surface
C-395
C-396
Top of rock
Geologic Fault
EXCAVATION PROFILE ALONG EARTH EMBANKME
EXCAVATION PROFILE ALONG SADDI



EXCAVATION PROFILE ALONG EARTH EMBANKMENT - GEORGIA

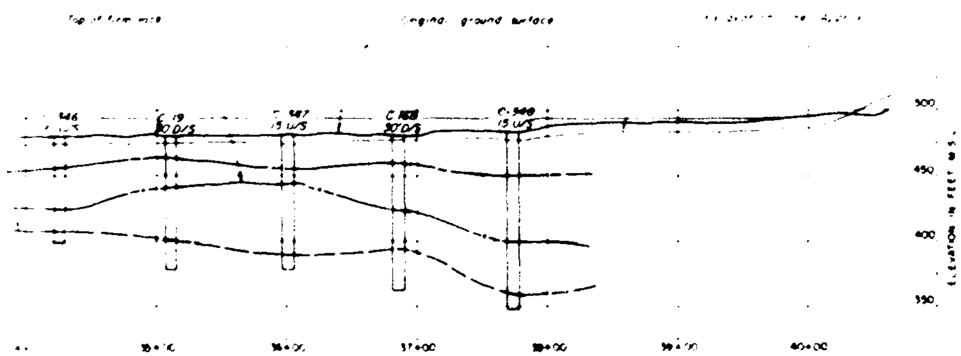


PROFILE ALONG EARTH EMBANKMENTS - GEORGIA EAST AND SOUTH CAROLINA

LEGEND

- Earthwork
- Original ground surface
- Top of rock
- Top of firm rock
- Top of sound rock
- Excavation line

NOTE: For geologic information pertaining to borings, see Plates 60 and 61



EXCAVATION PROFILE ALONG SADDLE DIKE - SOUTH CAROLINA

REVISIONS

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
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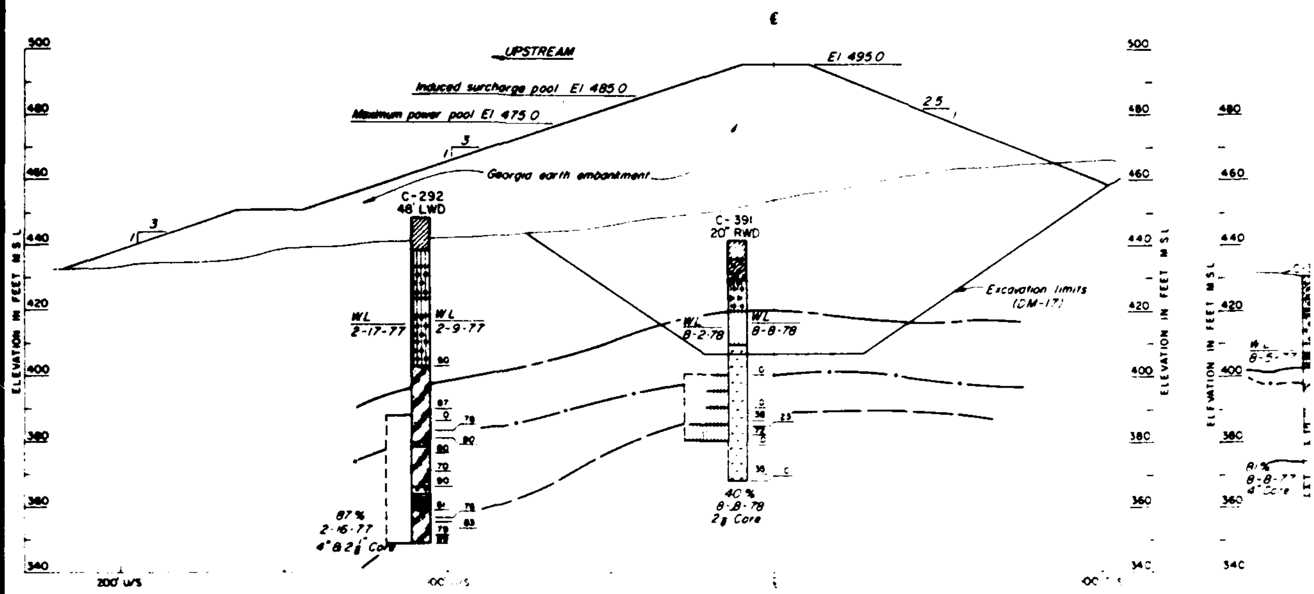
SAVANNAH, GEORGIA

FOUNDATION REPORT - EMBANKMENTS
EXCAVATION PROFILES
GEORGIA & SOUTH CAROLINA
EARTH EMBANKMENTS & SADDLE DIKE
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

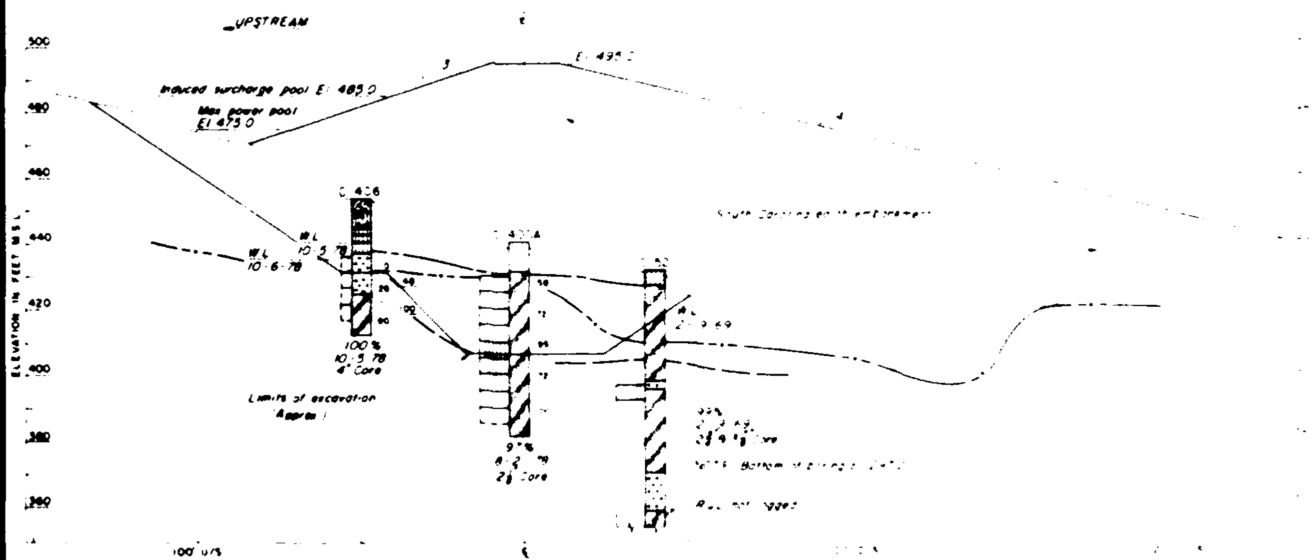
PLATE 62

2



GEOLOGIC SECTION AT STA 5+30 (A-A)

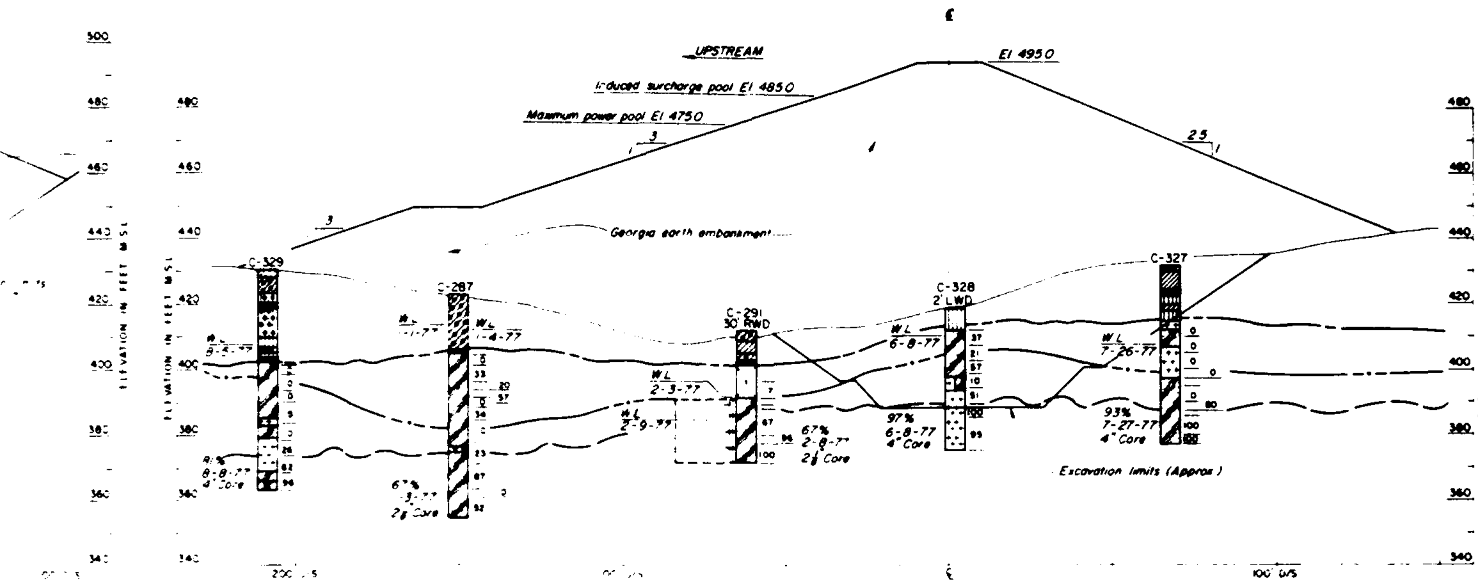
NOTE: VERTICAL EXAGGERATION 2.5X
SCALE 1"=20' VERT
1"=50' HORIZ



GEOLOGIC SECTION AT STA 44+00 (C-C)

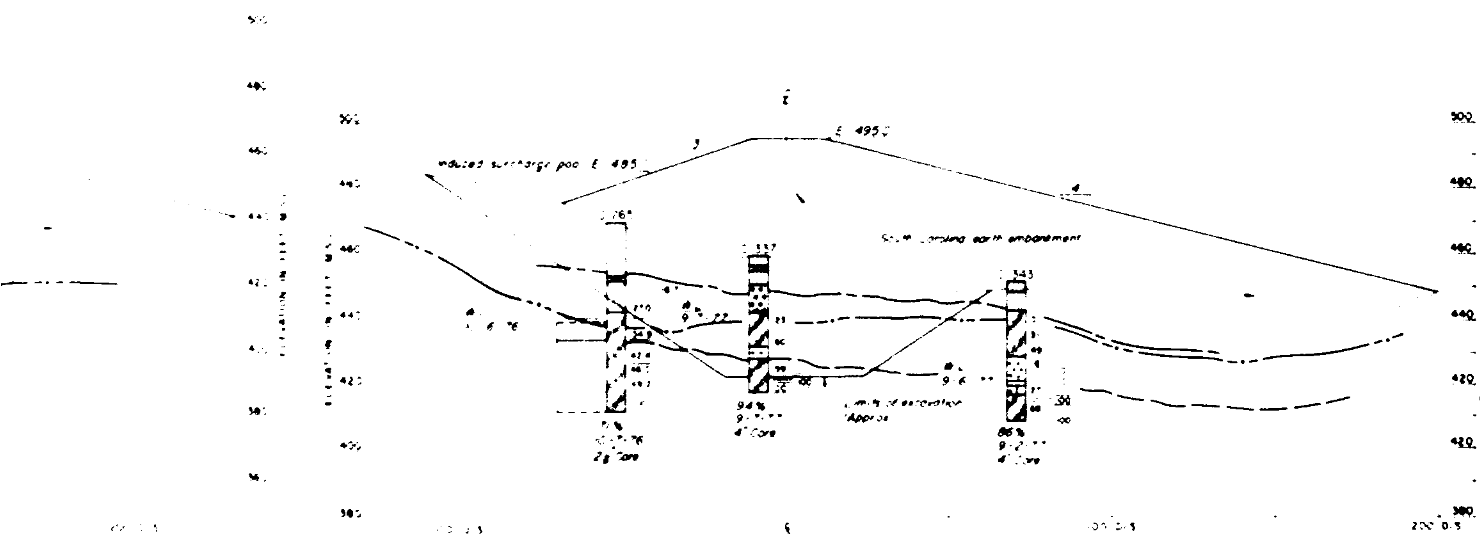
NOTES

1. Geologic logs of borings are maintained in the files of SASEN-GG
2. For locations of sections in plan view, refer to plates 58 and 59



GEOLOGIC CROSS SECTION AT STA 7+46 (B-B)

NOTE: VERTICAL EXAGGERATION = 2.5 X
SCALE: 1:20 VERT.
1:50 HORIZ.



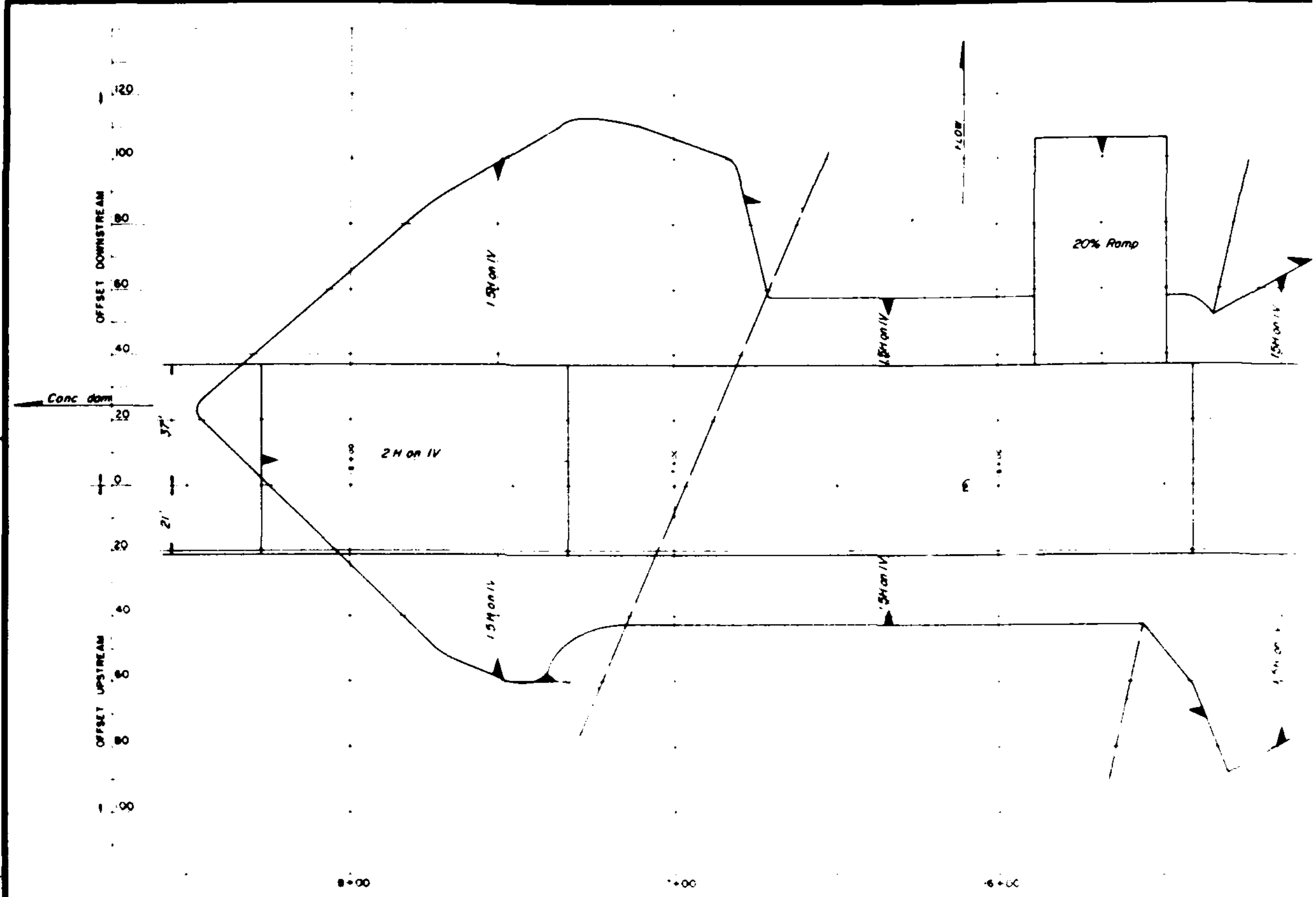
GEOLOGIC SECTION AT STA 45+05 (D-D)

NOTE: VERTICAL EXAGGERATION = 2.5 X
SCALE: 1:20 VERT.
1:50 HORIZ.

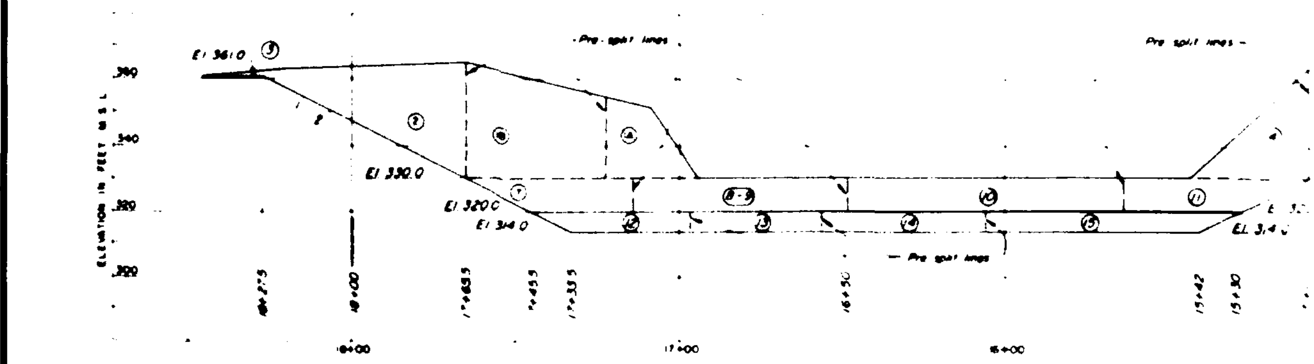
REVISIONS

U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
GEOLOGIC CROSS SECTIONS
GA & S CAROLINA EARTH EMBANKMENTS
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER, GEORGIA AND SOUTH CAROLINA

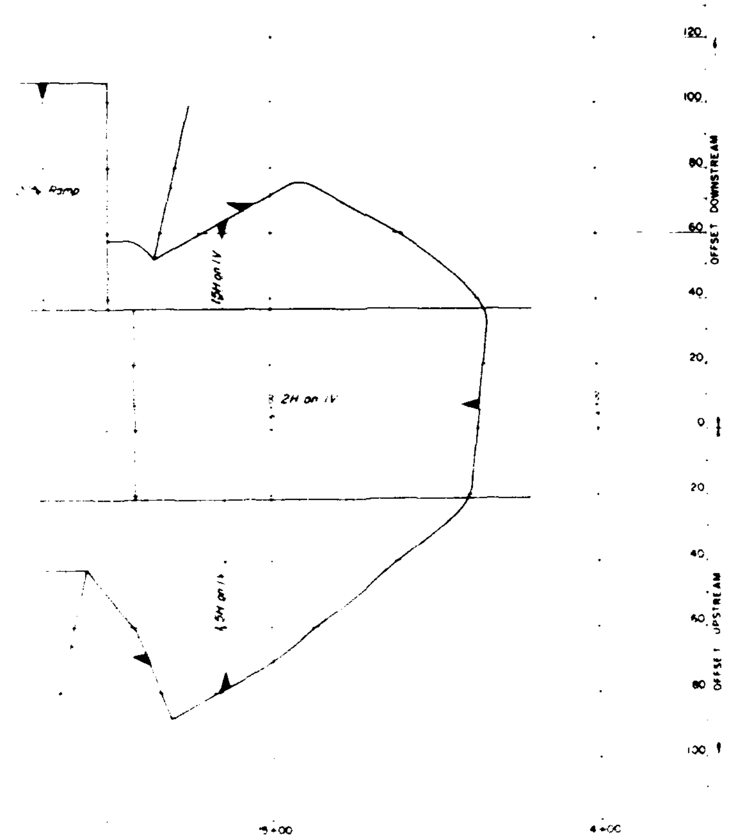
63



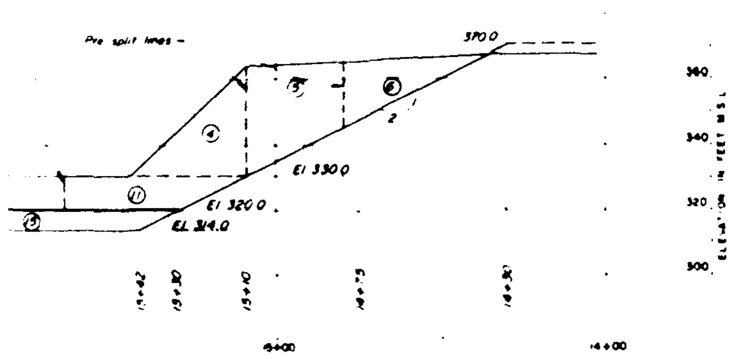
CORE TRENCH EXCAVATION (GEORGIA EMBANKMENT)
PLAN



PROFILE ALONG CENTERLINE (GEORGIA EMBANKMENT)



(EMBANKMENT)



(EMBANKMENT)

CORE TRENCH BLASTING			
SHOT NO	BURDEN SPAC	HOLE DIA	LOCATION
1A	7' x 5'	3½"	16+81.5 - 17+30.5
1B	7' x 5'	3½"	17+23.5 - 17+65.5
2	5' x 7'	3½"	17+65.5 - 18+05.5
3	5' x 7'	3½"	18+10.5 - 18+80.5
4	5' x 7'	3½"	15+10 - 15+35
5	5' x 7'	3½"	14+70 - 15+05
6	5' x 4'	2½"	14+45 - 14+65
7	5' x 7'	3½"	17+20.5 - 17+60.5
8-9	5' x 7'	3½"	16+50.5 - 17+15.5
10	5' x 7'	3½"	15+80.5 - 16+45.5
11	5' x 7'	3½"	15+05.5 - 15+75.5
12	4' x 4'	2½"	16+97.5 - 17+37.5
13	4' x 4'	2½"	16+57.5 - 16+95.5
14	4' x 4'	2½"	15+97.5 - 16+49.5
15	4' x 4'	2½"	15+37.5 - 15+93.5
RAMP	5' x 5'	3½"	See planview

NOTES:

POWDER Hercules Unigel 2⅜", 2½", 2" x 16"
POWDER FACTOR 1.1 lb/cy
STEMMING 3' Minimum
SUBDRILL 1' Maximum
MINIMUM DEPTH OF HOLE 4'

SYMBOL	DATE	DESCRIPTION	APP'D	BY

REVISIONS

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

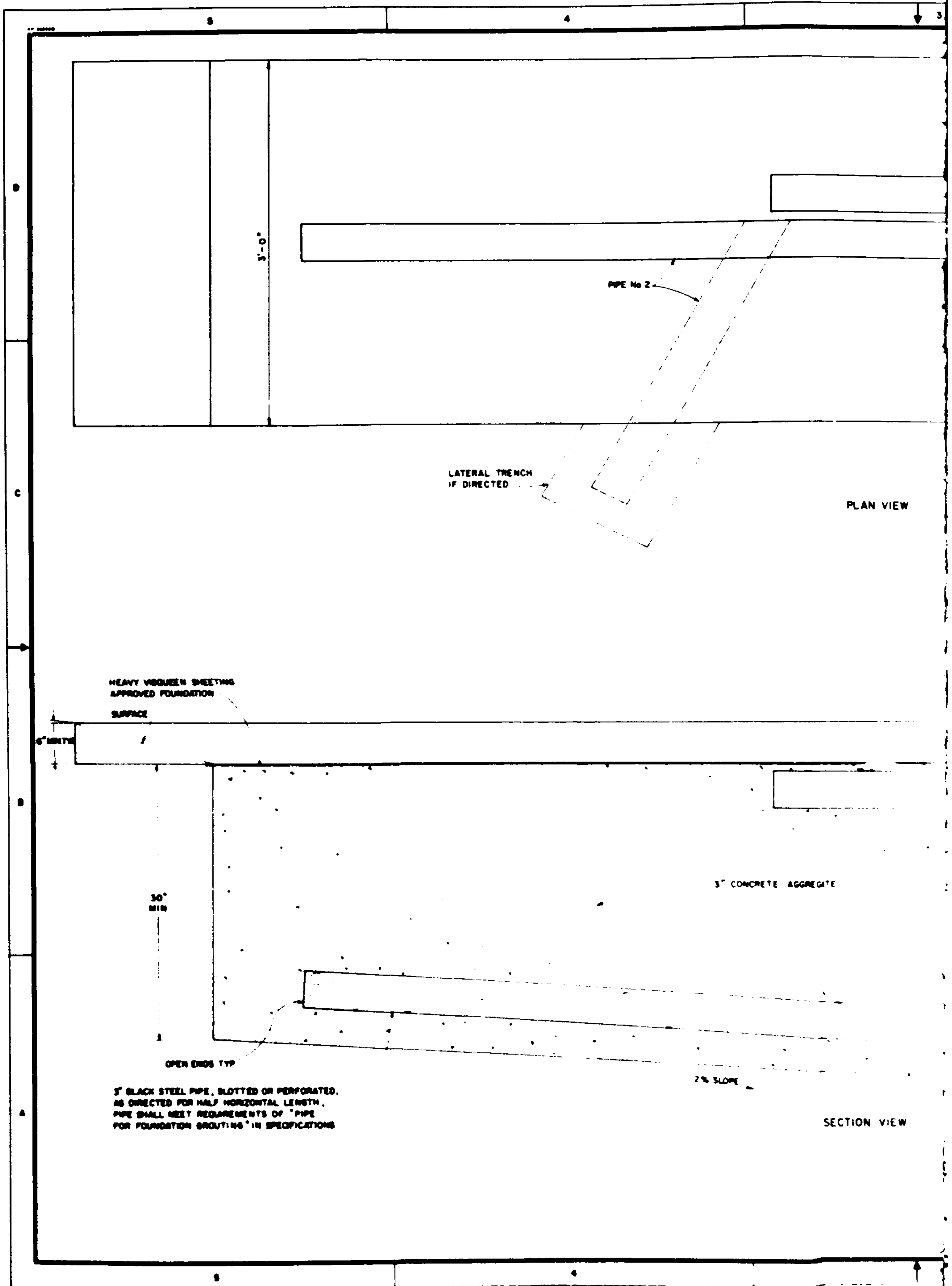
FOUNDATION REPORT
EMBANKMENTS
CORE TRENCH BLASTING
EARTH EMBANKMENTS
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE	CONTRACT NO.	FILE NO.	PLATE
F			64

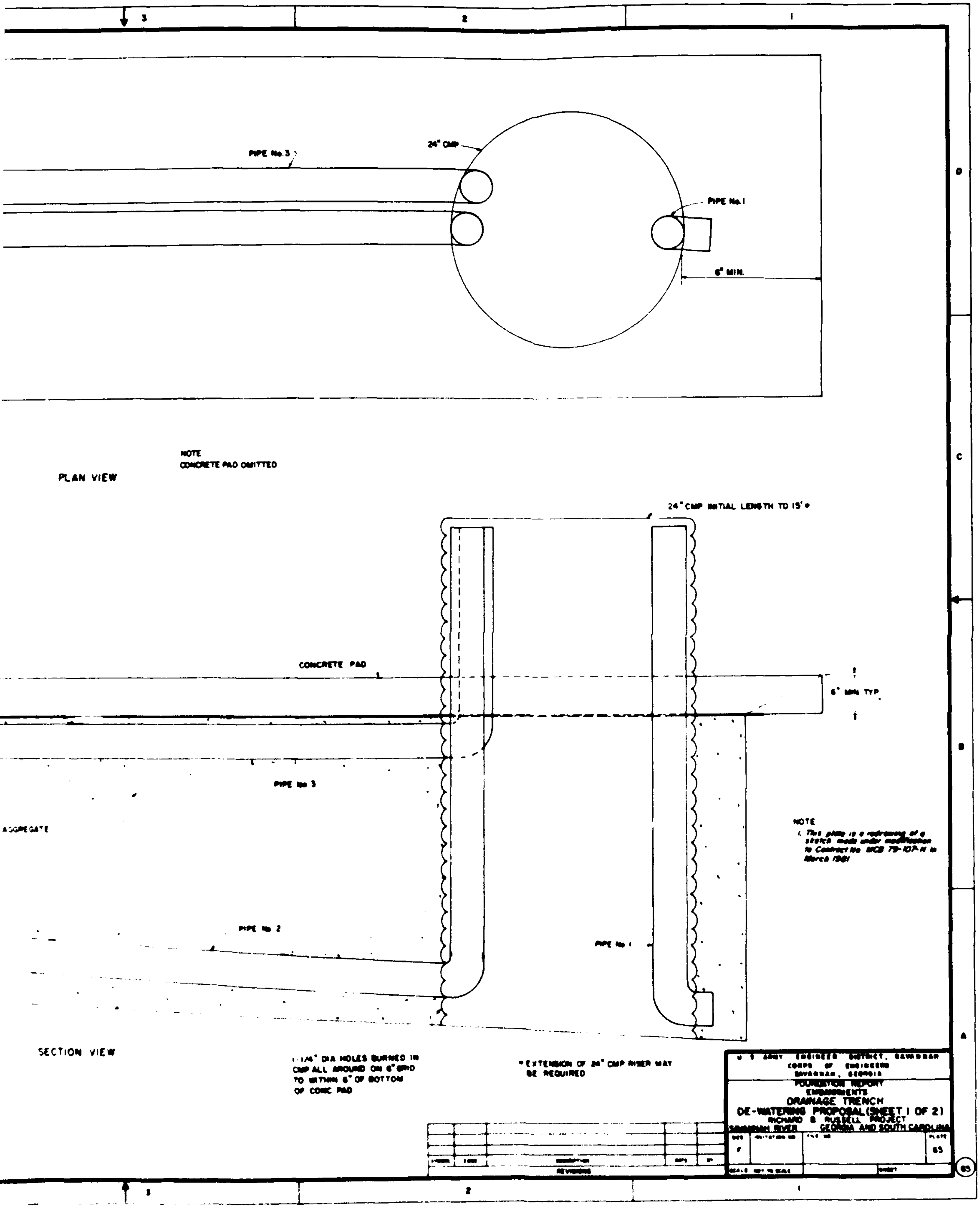
SCALE AS SHOWN | SHEET

64

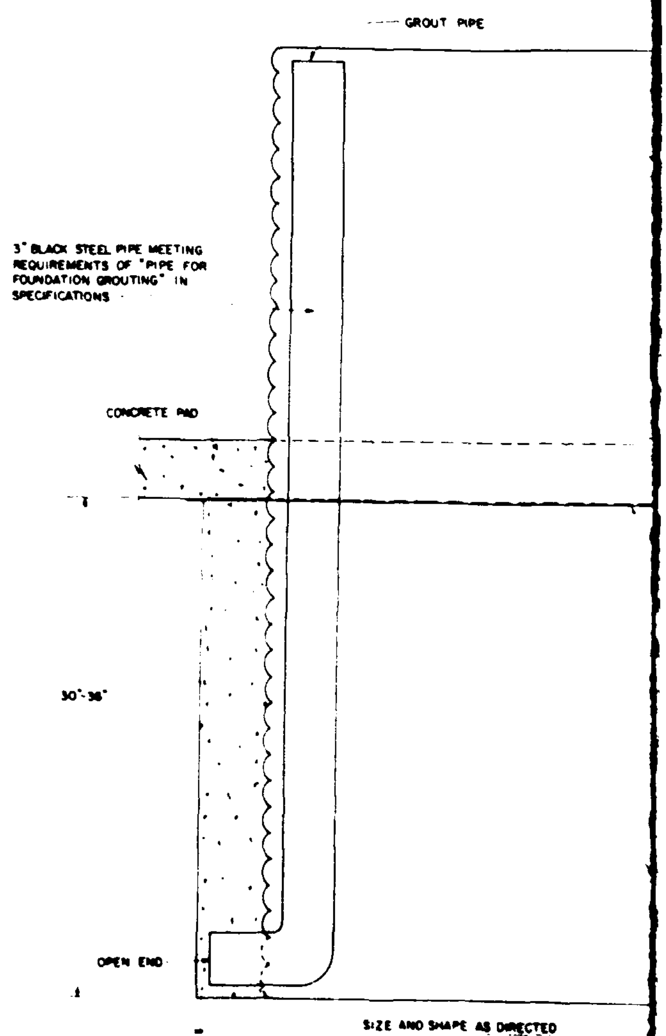
2



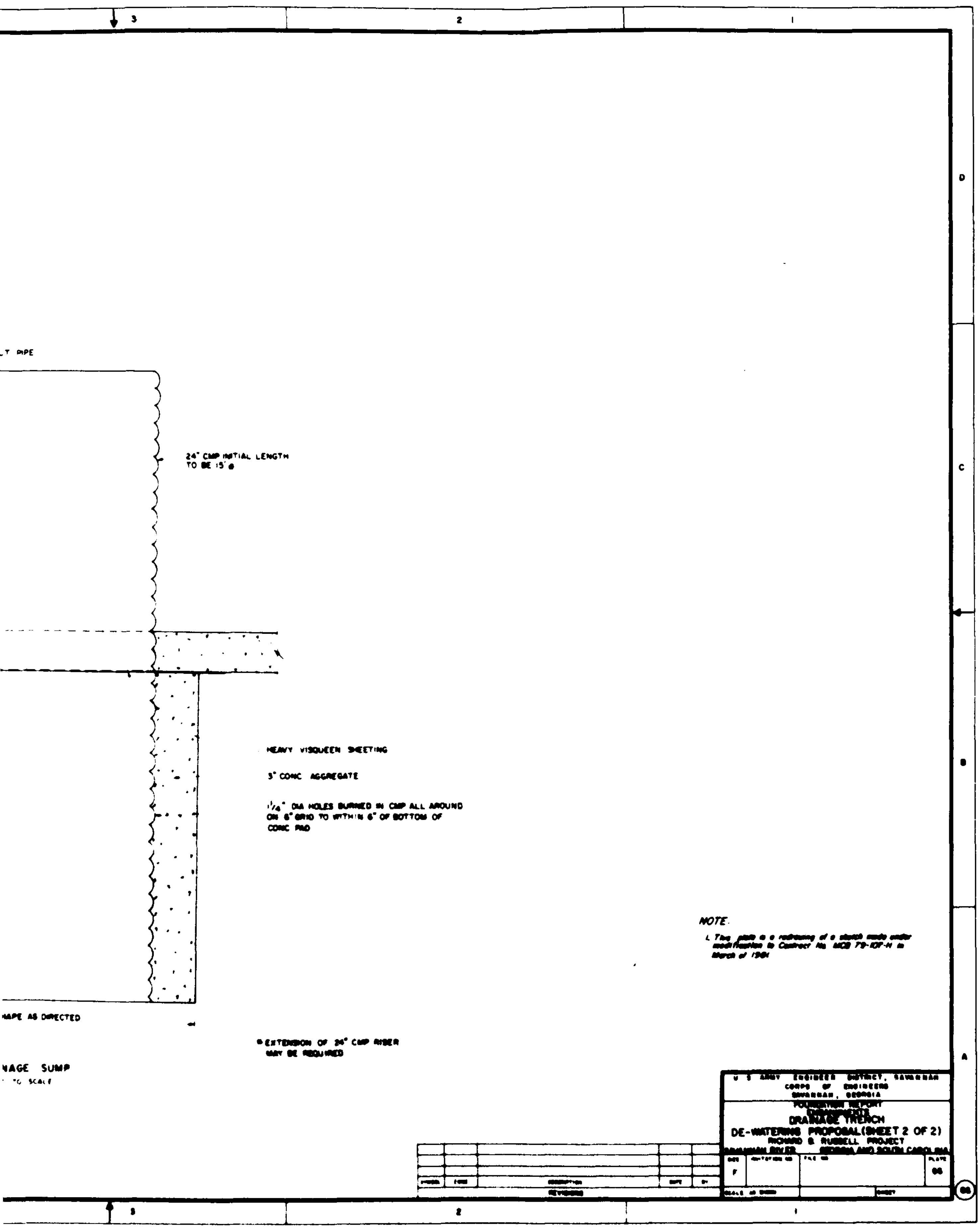
3'-0"
PIPE No 2
LATERAL TRENCH
IF DIRECTED
PLAN VIEW
HEAVY VISQUEEN SHEETING
APPROVED FOUNDATION
SURFACE
6" MIN TYP
30"
MIN
3" CONCRETE AGGREGATE
OPEN ENDS TYP
3" BLACK STEEL PIPE, SLOTTED OR PERFORATED,
AS DIRECTED FOR HALF HORIZONTAL LENGTH,
PIPE SHALL MEET REQUIREMENTS OF "PIPE
FOR FOUNDATION GROUTING" IN SPECIFICATIONS
2% SLOPE
SECTION VIEW



PIPE No.3
24" CMP
PIPE No.1
6" MIN.
NOTE
CONCRETE PAD OMITTED
PLAN VIEW
24" CMP INITIAL LENGTH TO 15'*
CONCRETE PAD
6" MIN TYP.
PIPE No 3
AGGREGATE
PIPE No 2
PIPE No 1
NOTE
1. This plate is a redrawing of a sketch made under modification to Contract No. MCB 79-107-H in March 1981
SECTION VIEW
1-1/4" DIA HOLES BURNED IN CMP ALL AROUND ON 6" GRID TO WITHIN 6" OF BOTTOM OF CONC PAD
*EXTENSION OF 24" CMP RISER MAY BE REQUIRED
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
DRAINAGE TRENCH
DE-WATERING PROPOSAL (SHEET 1 OF 2)
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
PLATE
63
SCALE NOT TO SCALE
REVISIONS



DRAINAGE SUMP
NOT TO SCALE



NOTE:

1. This plate is a redrawing of a sketch made under modification to Contract No. MCB 79-IOF-H in March of 1981

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
DRAINAGE TRENCH
DE-WATERING PROPOSAL (SHEET 2 OF 2)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

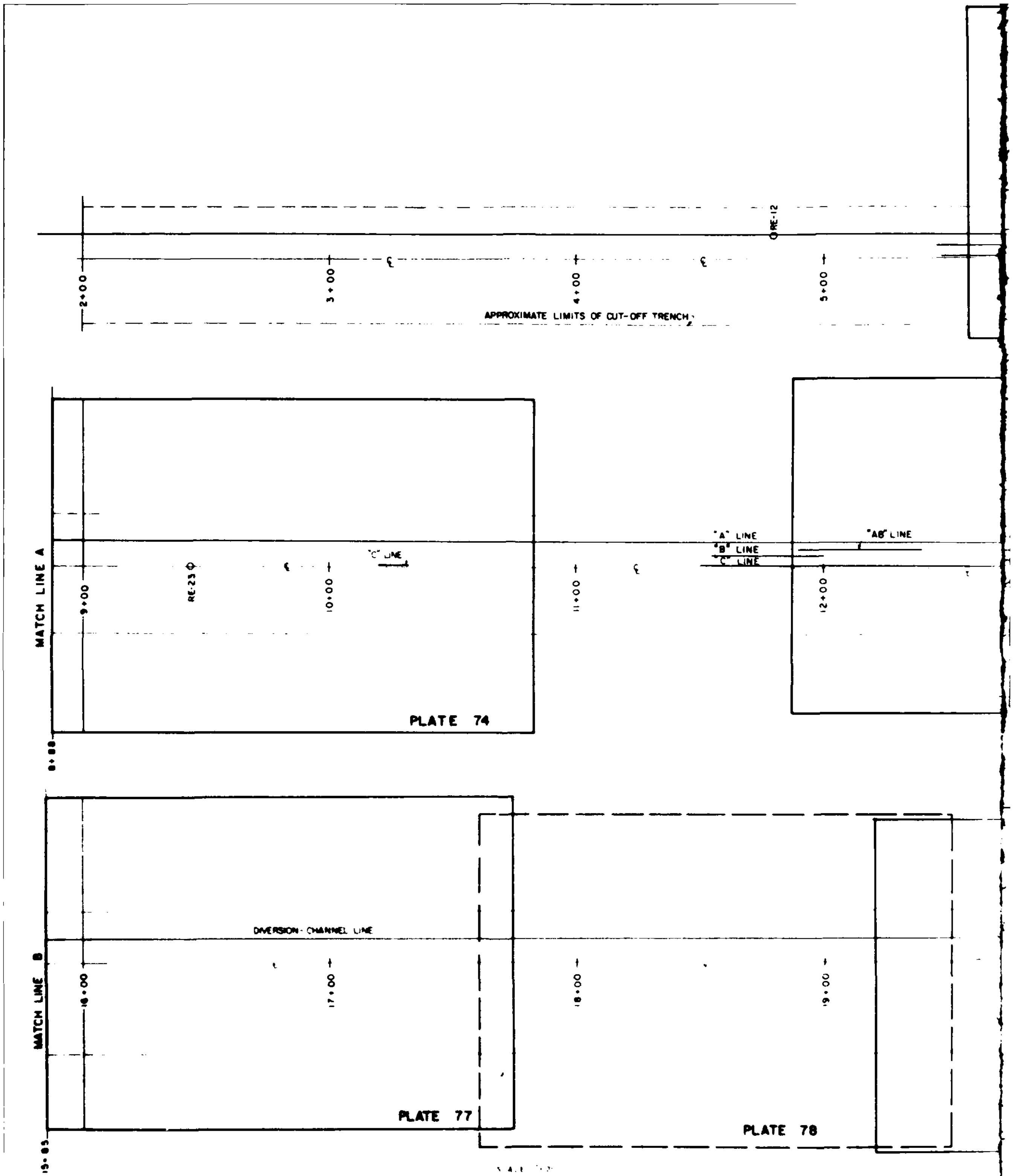
SIZE	INVITATION NO.	FILE NO.	PLATE
F			66

SCALE AS SHOWN SHEET

SYMBOL	ZONE	DESCRIPTION	DATE	BY
		REVISIONS		

66

2



NOTES

1 CURVATURE OF EMBANKMENT NOT SHOWN

CURVE DATA

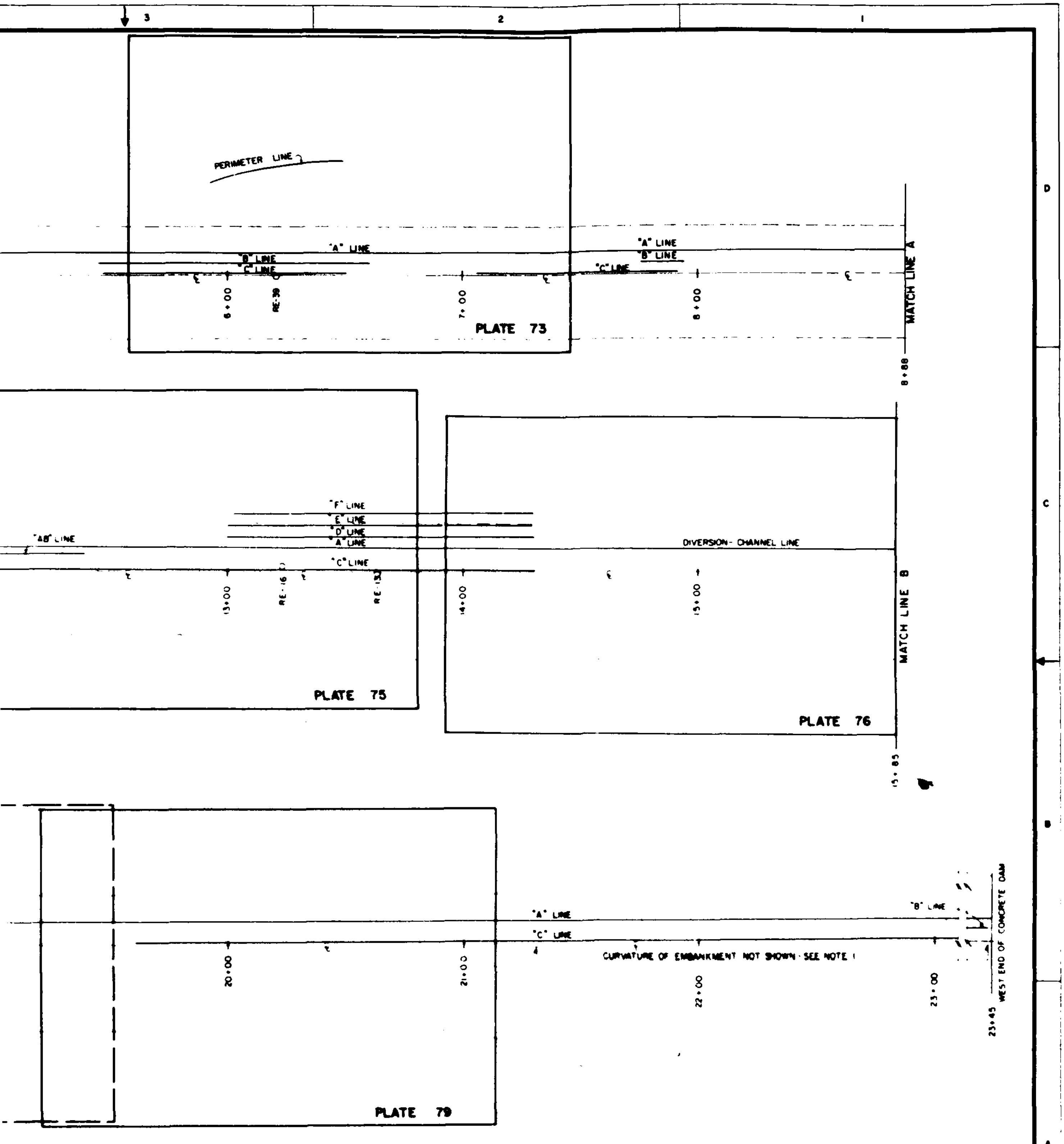
PI = 22+53.21

D = 19.94°

R = 287.32

T = 122.02

LC = 230.79



...MENT NOT SHOWN

2. EXPLORATION BORING DENOTED BY ○ RE-12
3. "A" LINE DENOTES GROUT CURTAIN
4. MAPS DO NOT SHOW AS-BUILT LIMITS OF SHOTCRETE AND DENTAL CONCRETE. REFER TO EMBANKMENT CRITERIA AND PERFORMANCE REPORT

REVISIONS

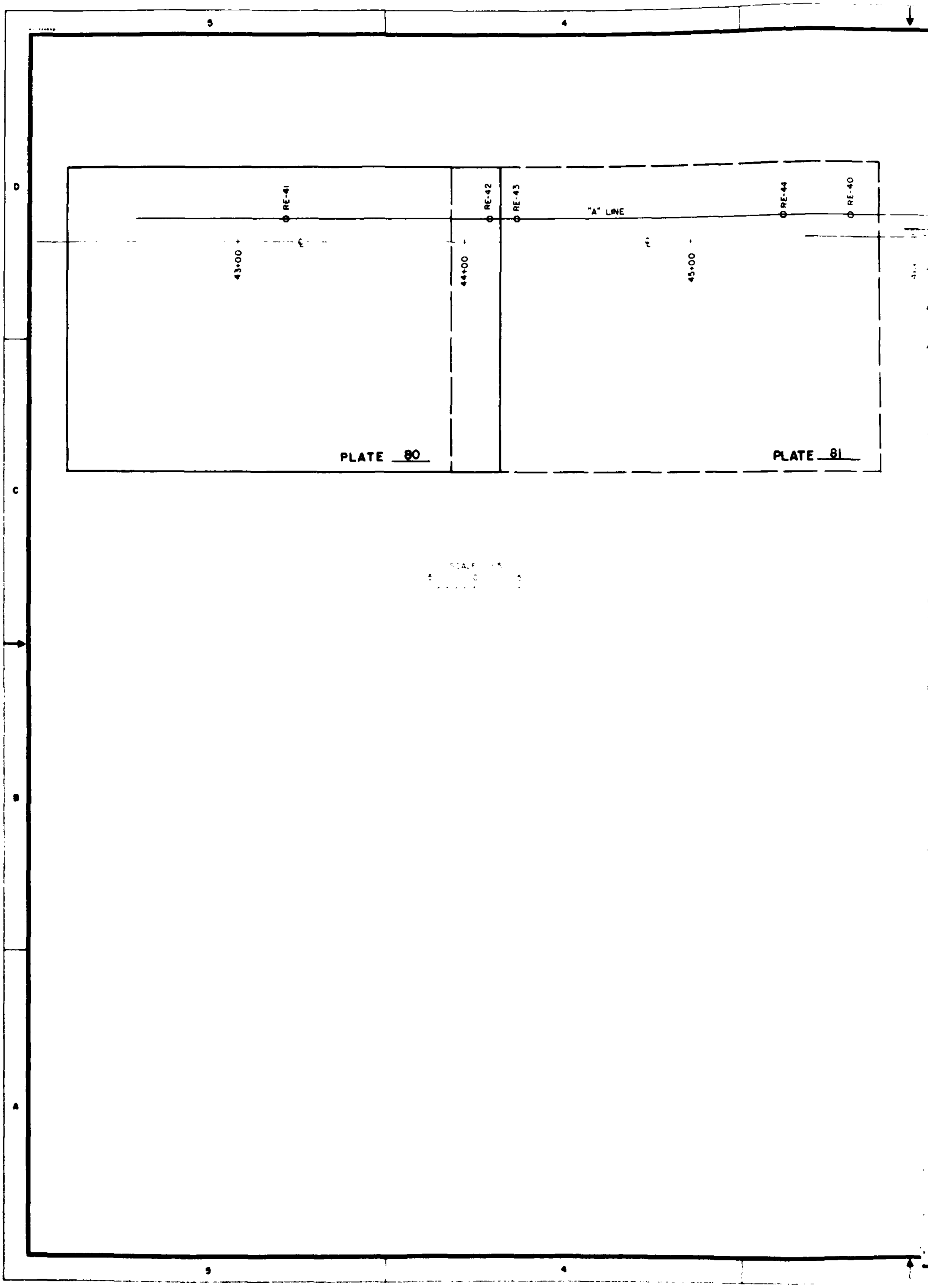
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

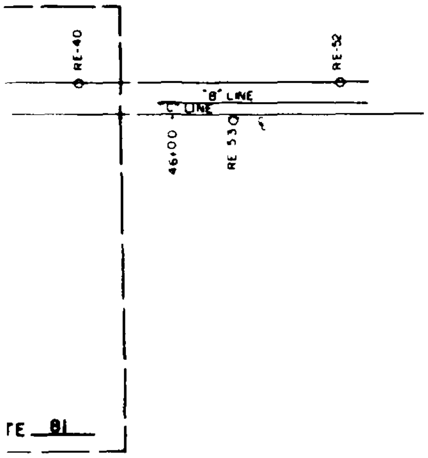
FOUNDATION REPORT
EMBANKMENTS
FOUNDATION MAP COVERAGE AND
GROUT CURTAIN LOCATIONS (SHEET 1 OF 2)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

67

(67)

2


5
4
D
C
B
A
RE-41
RE-42
RE-43
"A" LINE
RE-44
RE-40
43+00
44+00
45+00
PLATE 80
PLATE 81



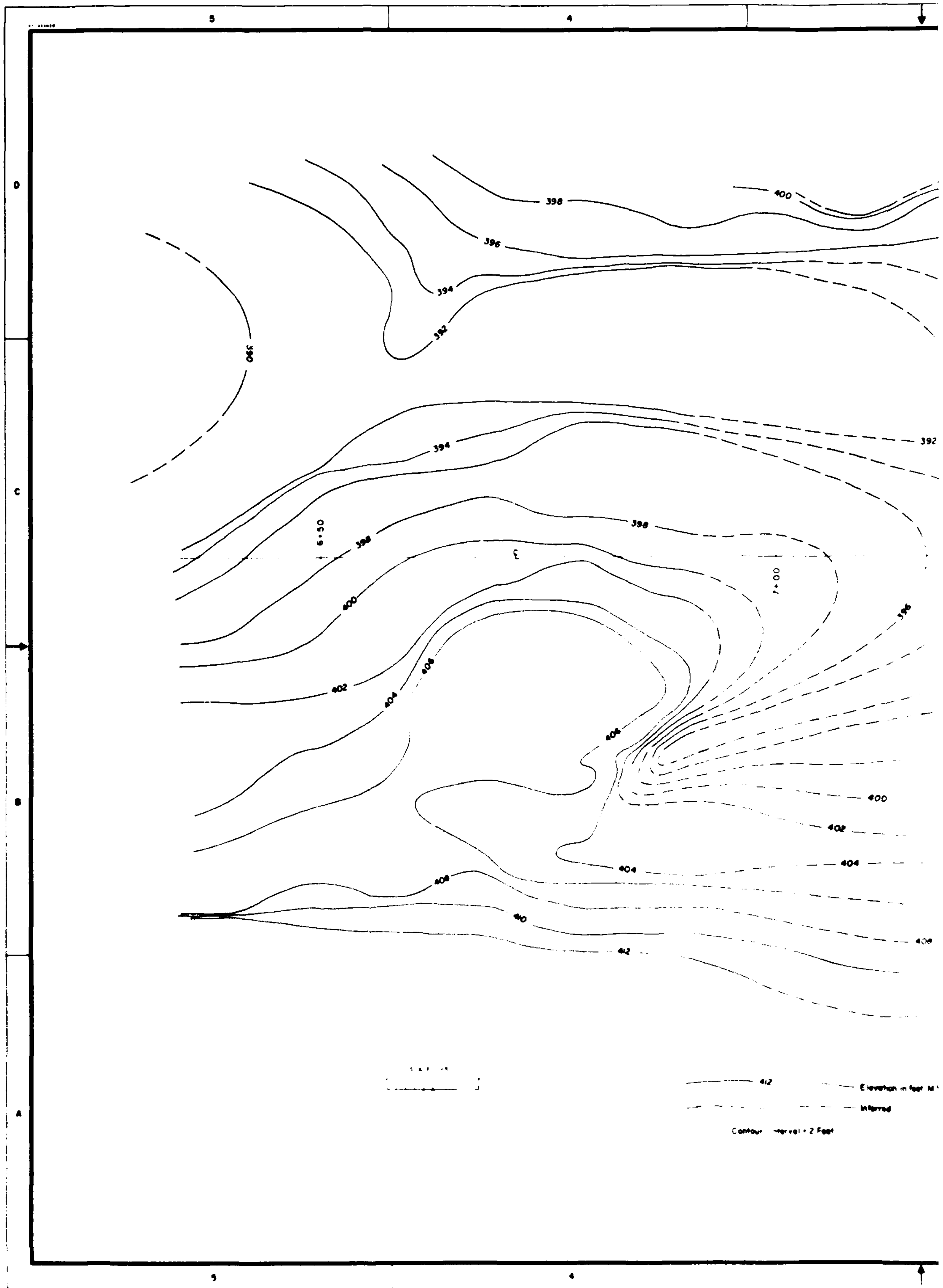
NOTES

1. CURVATURE OF EMBANKMENT NOT SHOWN
   CURVE DATA:
   PI = 46+25.54
   D = 15°
   R = 383.06'
   T = 198.84'
   LC = 366.67'
2. EXPLORATION BORING DENOTED BY ○ RE-2
3. "A" LINE ___ DENOTES GROUT CURTAIN
4. MAPS DO NOT SHOW "AS-BUILT" LIMITS OF OF SHOTCRETE AND DENTAL CONCRETE. REFER TO EMBANKMENT CRITERIA AND PERFORMANCE REPORT

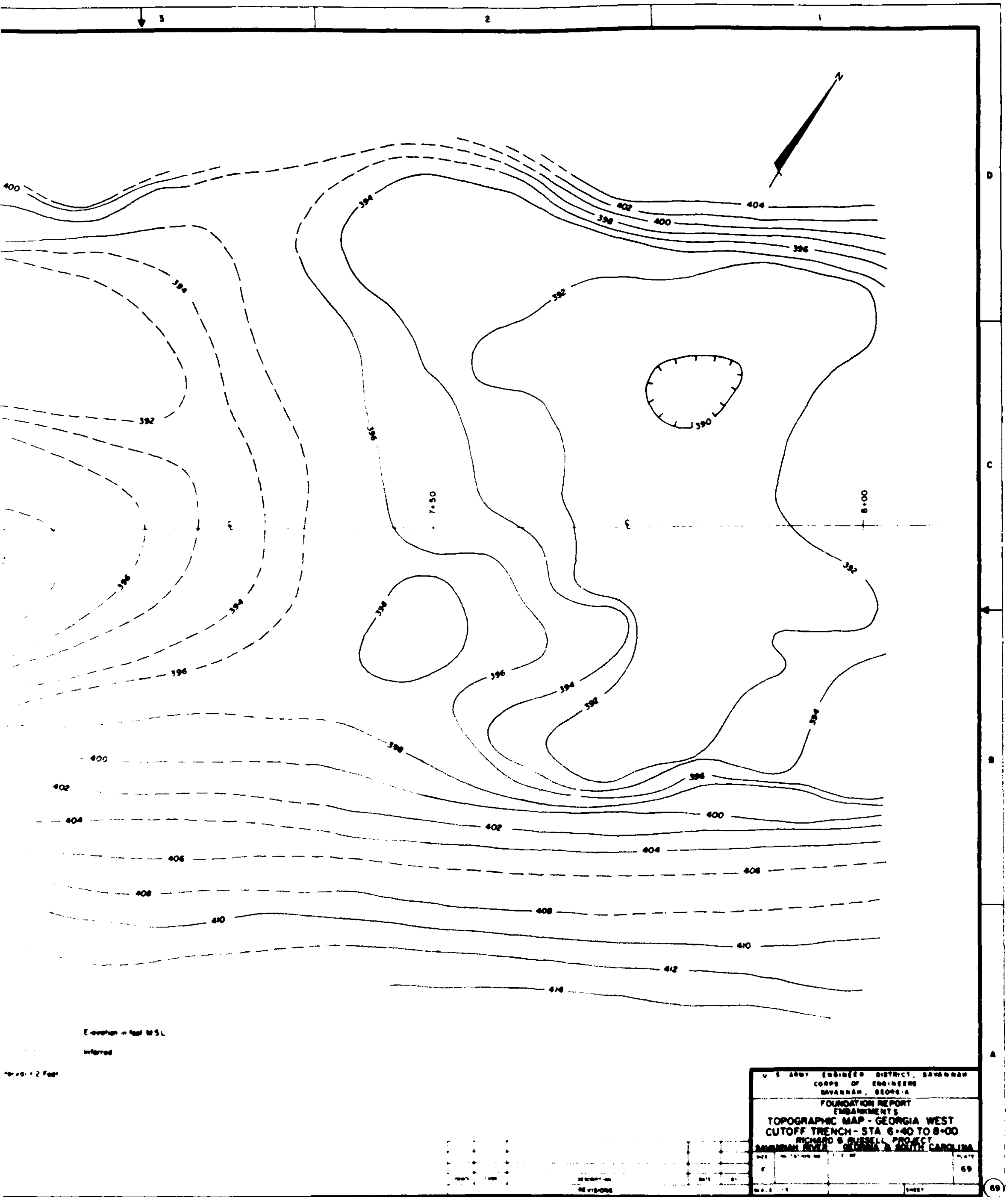
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
EMBANKMENTS
FOUNDATION MAP COVERAGE & GROUT CURTAIN LOCATIONS (SHEET 2 OF 2)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER — GEORGIA AND SOUTH CAROLINA

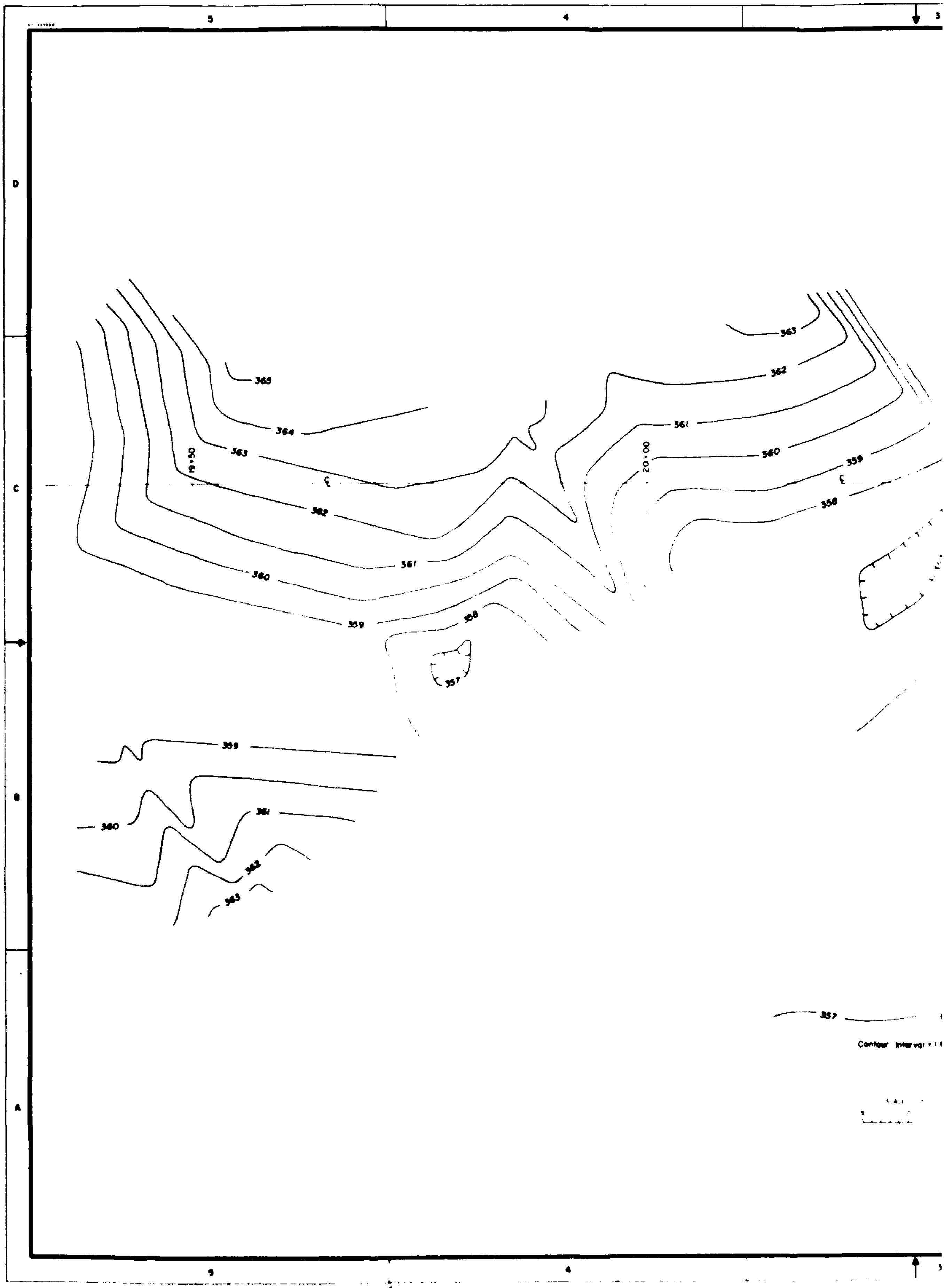
PLATE 68

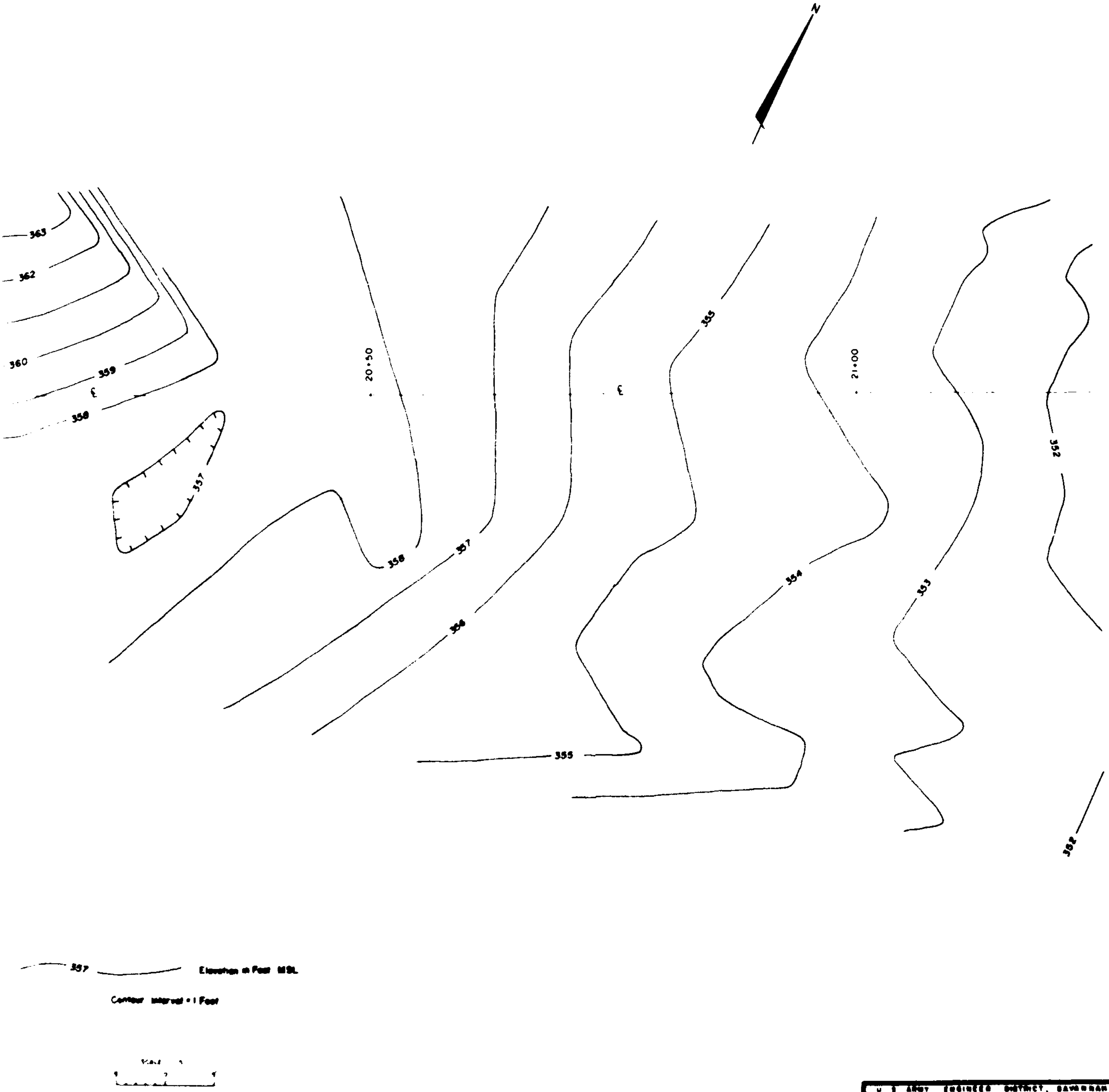


5
4
D
C
B
A
398
400
396
394
392
390
392
394
398
398
6+50
7+00
400
396
402
404
406
408
406
400
402
404
404
408
410
412
404
412
Elevation in feet M
Inferred
Contour interval = 2 Feet
5
4


N
400
402
404
396
398
394
392
390
7+50
8+00
406
408
410
412
414
Elevation in feet M.S.L.
Inferred
REVISIONS
U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
TOPOGRAPHIC MAP - GEORGIA WEST
CUTOFF TRENCH - STA 6+40 TO 8+00
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER, GEORGIA & SOUTH CAROLINA
69

2


5
4
3
D
C
B
A
365
364
363
362
361
360
359
358
357
19+50
20+00
363
362
361
360
359
358
359
360
361
362
363
357
Contour Interval



U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

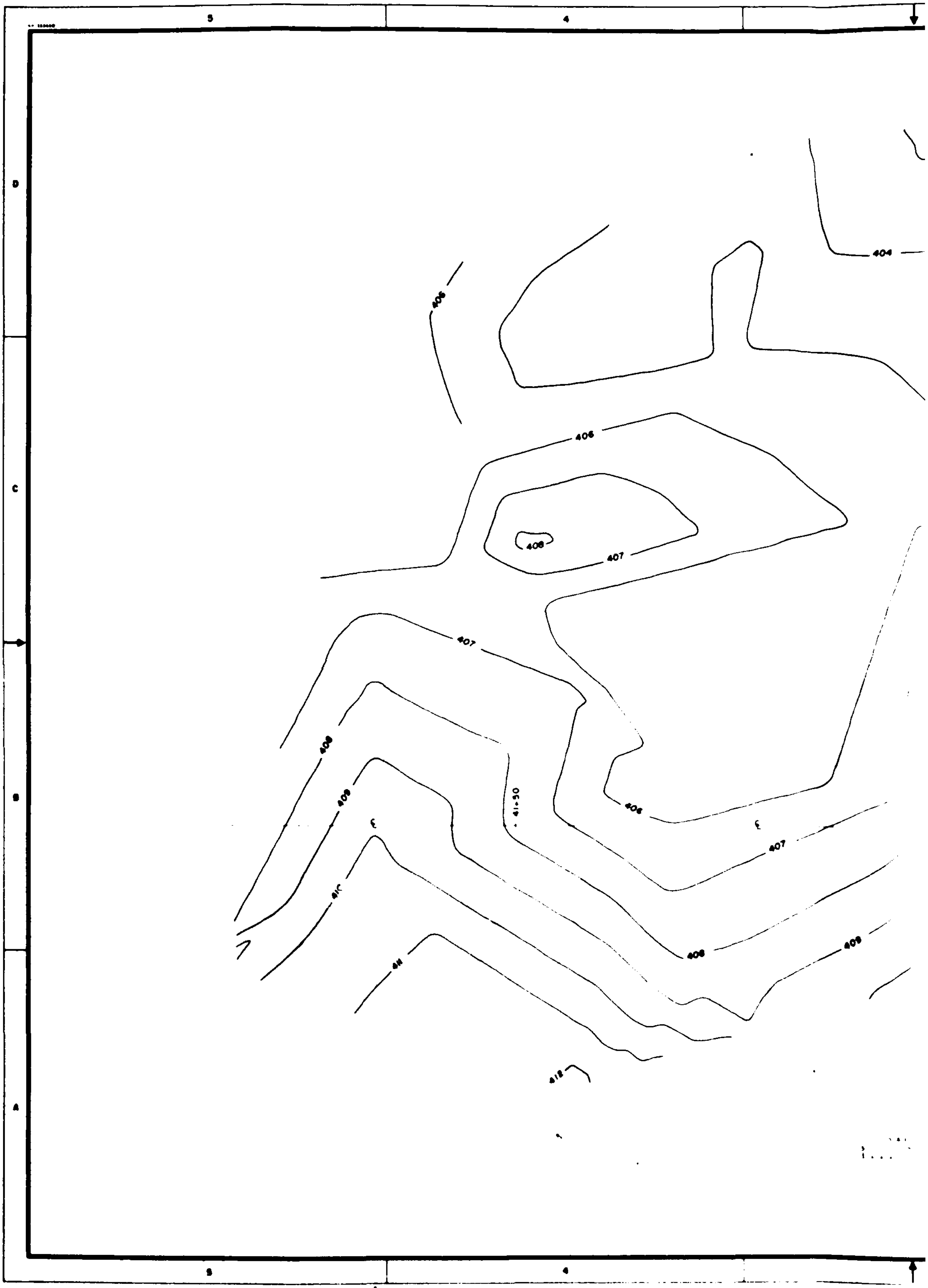
FOUNDATION REPORT
EMBANKMENTS

TOPOGRAPHIC MAP - GEORGIA EAST CUTOFF TRENCH - STA 19+35 TO 21+25

RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA & SOUTH CAROLINA

PLATE 70

2


5
4
D
C
B
A
404
406
406
407
408
407
406
408
409
41+50
407
408
409
411
412

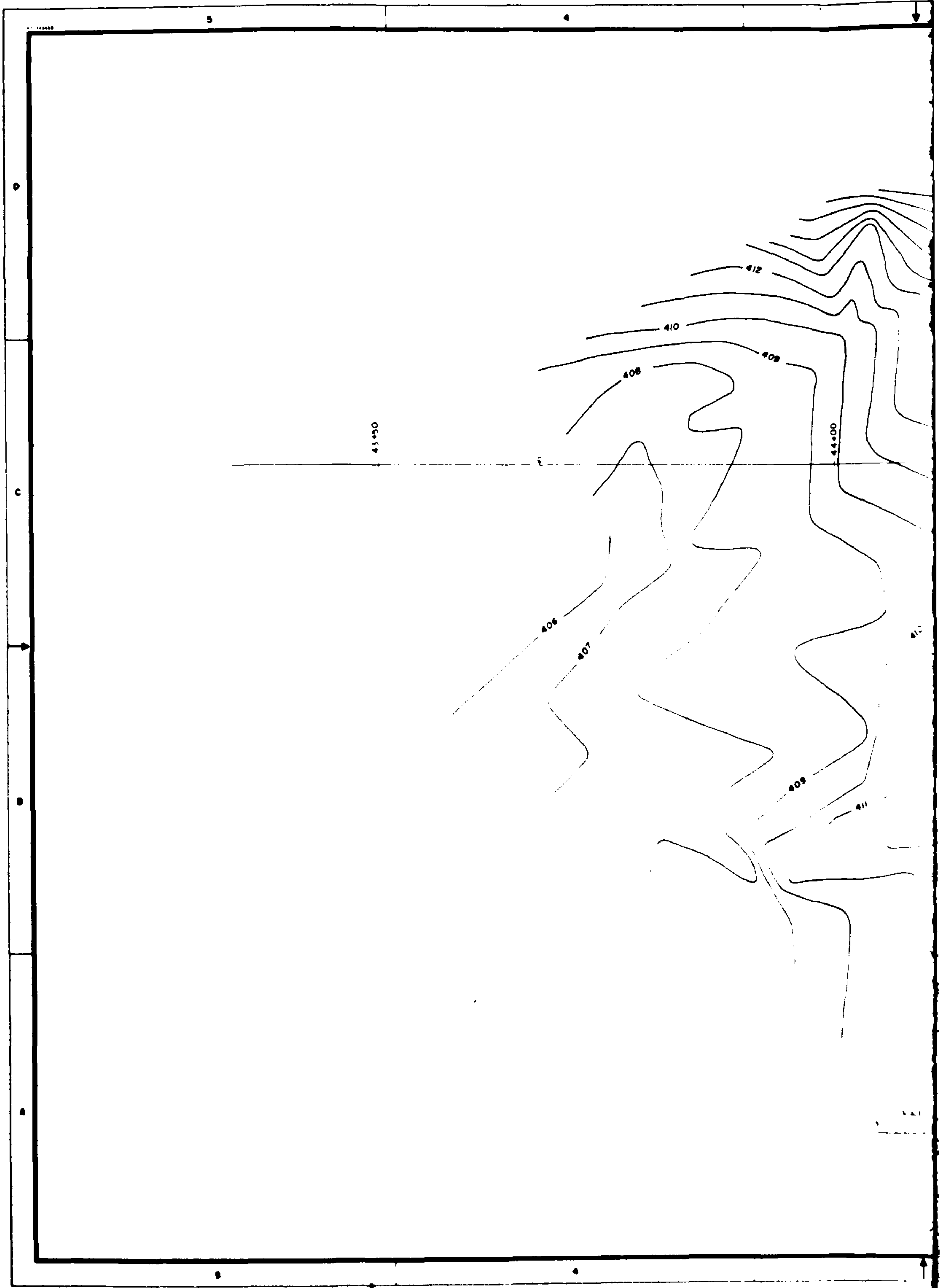
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
EMBANKMENTS
TOPOGRAPHIC MAP - SOUTH CAROLINA
CUTOFF TRENCH - STA. 41+20 TO 42+50
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA & SOUTH CAROLINA

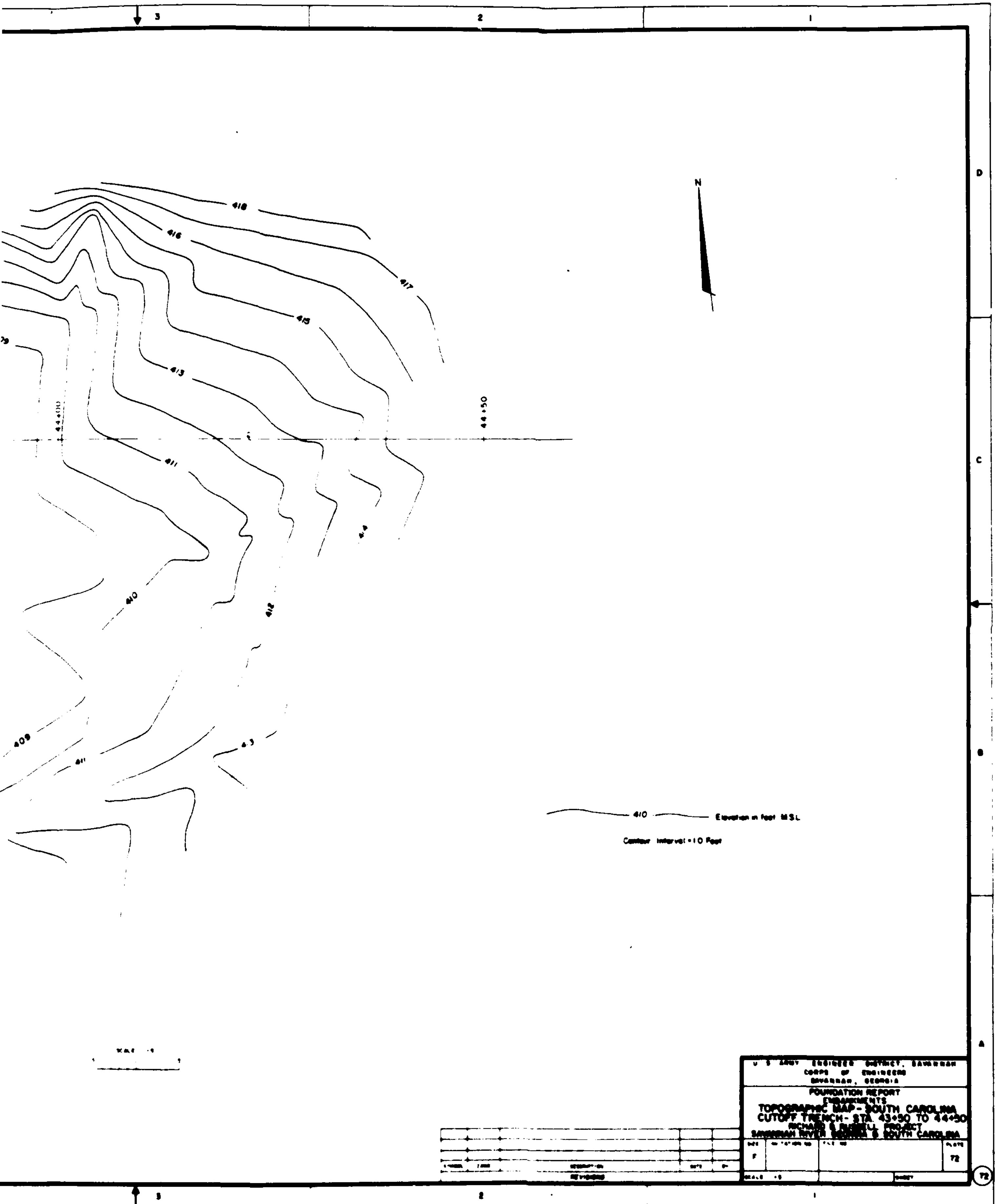
PLATE 71

71

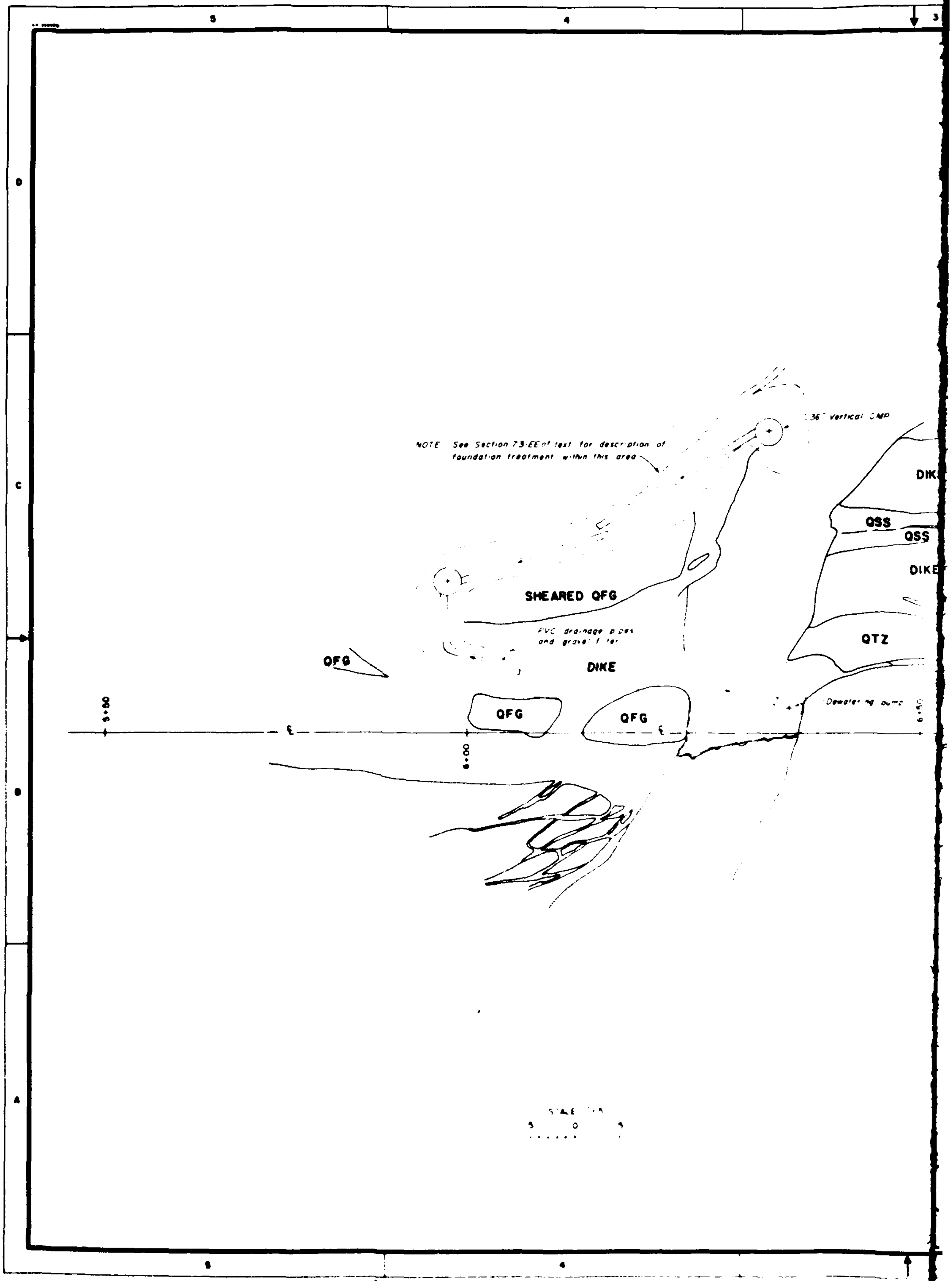
2



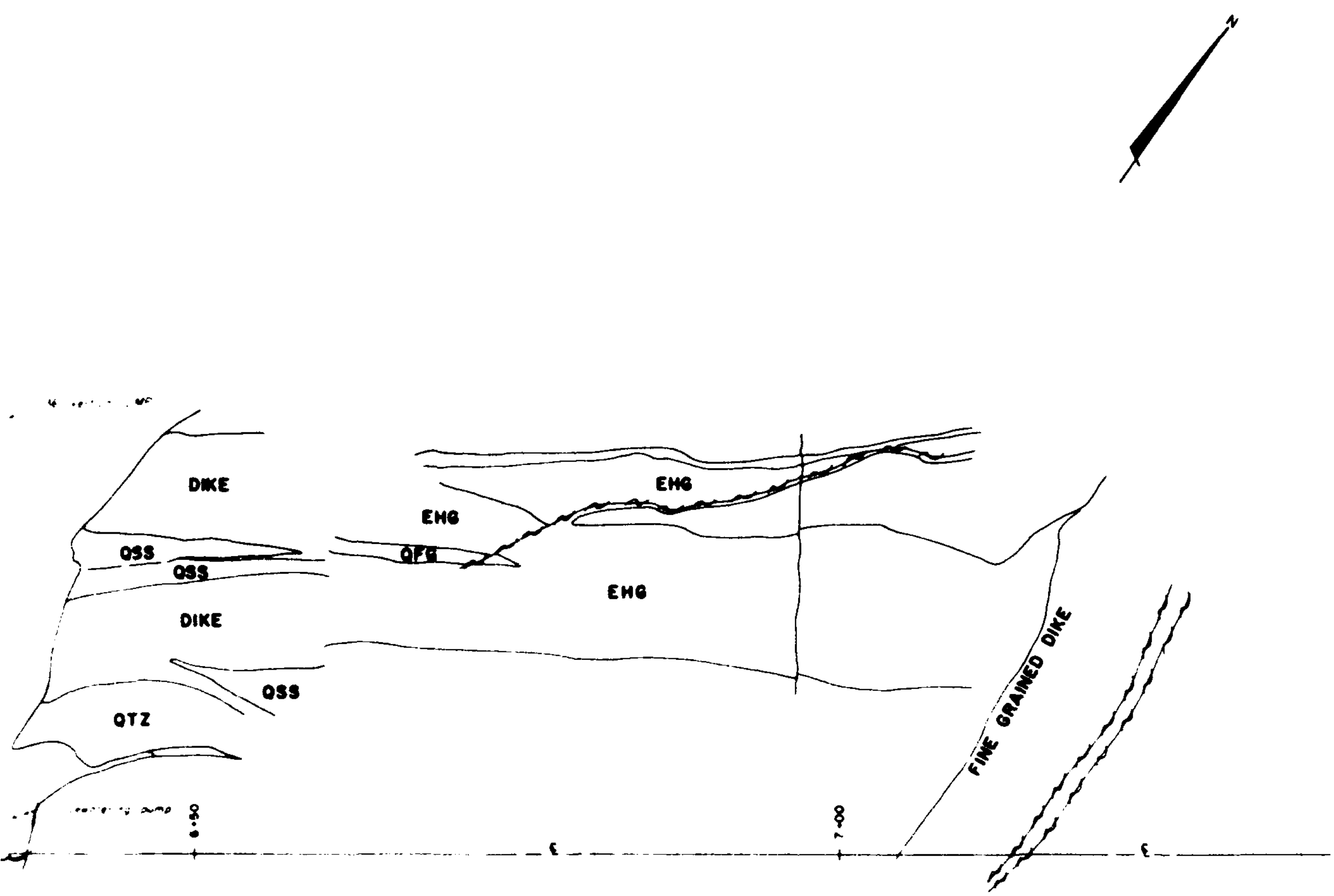
412
410
409
408
43+50
44+00
406
407
409
411



N
418
416
417
415
413
44+00
44+50
411
414
410
412
409
411
413
410
Elevation in feet M.S.L.
Contour Interval = 1.0 Foot
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
TOPOGRAPHIC MAP - SOUTH CAROLINA
CUTOFF TRENCH - STA. 43+50 TO 44+50
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA & SOUTH CAROLINA
REVISIONS
PLATE
72



5
4
3
D
C
B
A
36" Vertical CMP
NOTE See Section 73-EE of text for description of foundation treatment within this area
DIK
QSS
QSS
DIKE
SHEARED QFG
PVC drainage pipes and gravel filter
QTZ
QFG
DIKE
QFG
QFG
Dewatering pump
5+50
6+00
SCALE
5 0 5



LEGEND

- **QFG** - Quartz Feldspar Gneiss
- **EHG** - Epidote Hornblende Gneiss
- **QSS** - Quartz Sericite Schist
- **QTZ** - Quartz
- Sheared zone
- Fault
- Strike and dip of joints
- Vertical joint
- Limits of "Bathtub" Dewatering Area

REVISIONS

U S ARMY ENGINEER DISTRICT, SAVANNAH
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SAVANNAH, GEORGIA

FOUNDATION REPORT
EMBANKMENTS
GEOLOGIC MAP - GEORGIA WEST CUT-OFF
TRENCH-STA 5+50 TO 7+40
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

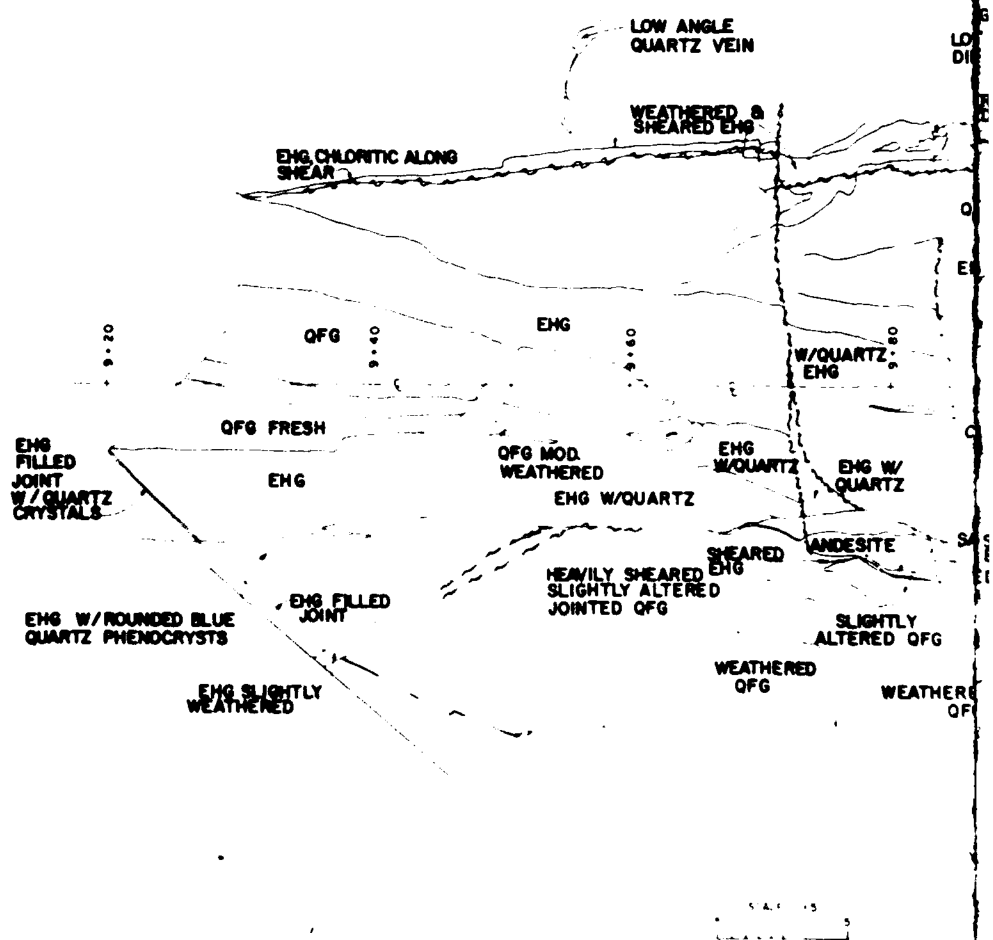
SIZE			PLATE
F			73

SCALE AS SHOWN

SHEET

73

2


LOW ANGLE QUARTZ VEIN
WEATHERED & SHEARED EHG
EHG, CHLORITIC ALONG SHEAR
EHG
QFG
9+20
9+40
9+60
9+80
W/QUARTZ EHG
QFG FRESH
EHG FILLED JOINT W/QUARTZ CRYSTALS
EHG
QFG MOD. WEATHERED
EHG W/QUARTZ
EHG W/QUARTZ
EHG W/ QUARTZ
SHEARED EHG
ANDESITE
HEAVILY SHEARED SLIGHTLY ALTERED JOINTED QFG
EHG W/ROUNDED BLUE QUARTZ PHENOCRYSTS
EHG FILLED JOINT
SLIGHTLY ALTERED QFG
WEATHERED QFG
EHG SLIGHTLY WEATHERED

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
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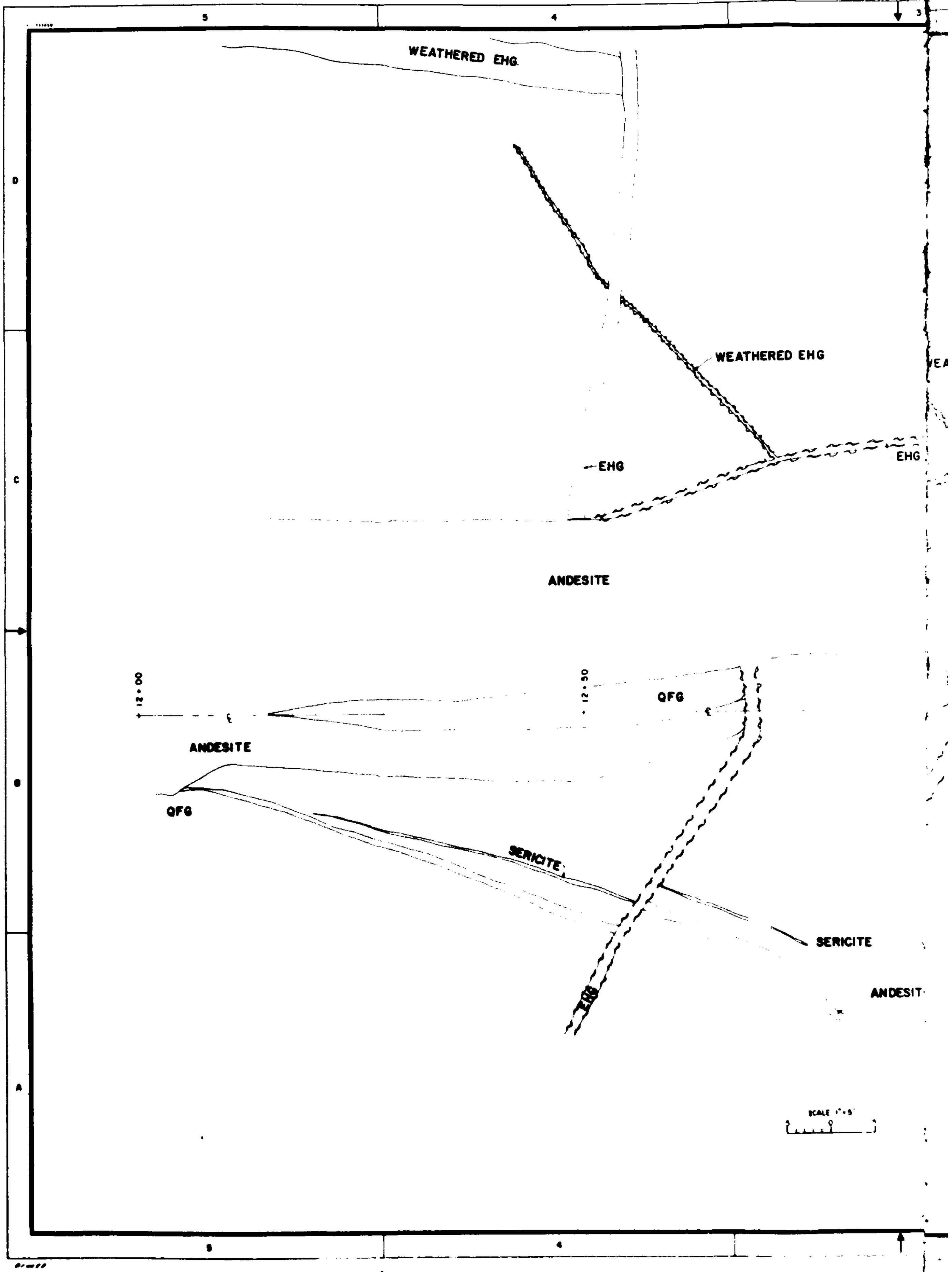
SAVANNAH, GEORGIA

FOUNDATION REPORT
EMBANKMENTS
GEOLOGIC MAP-GEORGIA WEST CUT-OFF TRENCH-STA 9+20 TO 10+50
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER — GEORGIA AND SOUTH CAROLINA

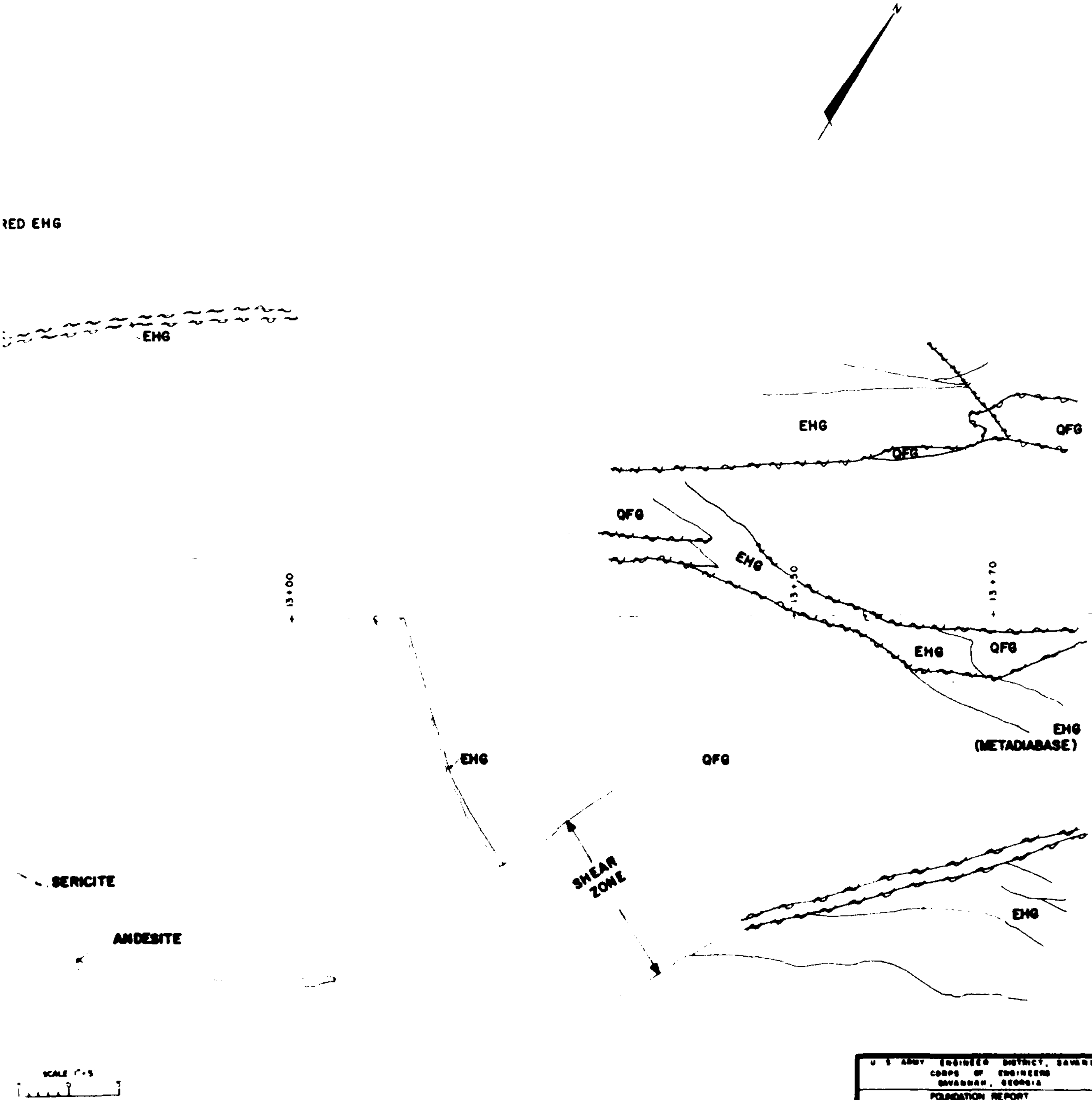
PLATE 76

SCALE AS SHOWN

2



WEATHERED EHG.
WEATHERED EHG
EHG
EHG
ANDESITE
12+00
12+50
QFG
ANDESITE
QFG
SERICITE
SERICITE
EHG
ANDESIT



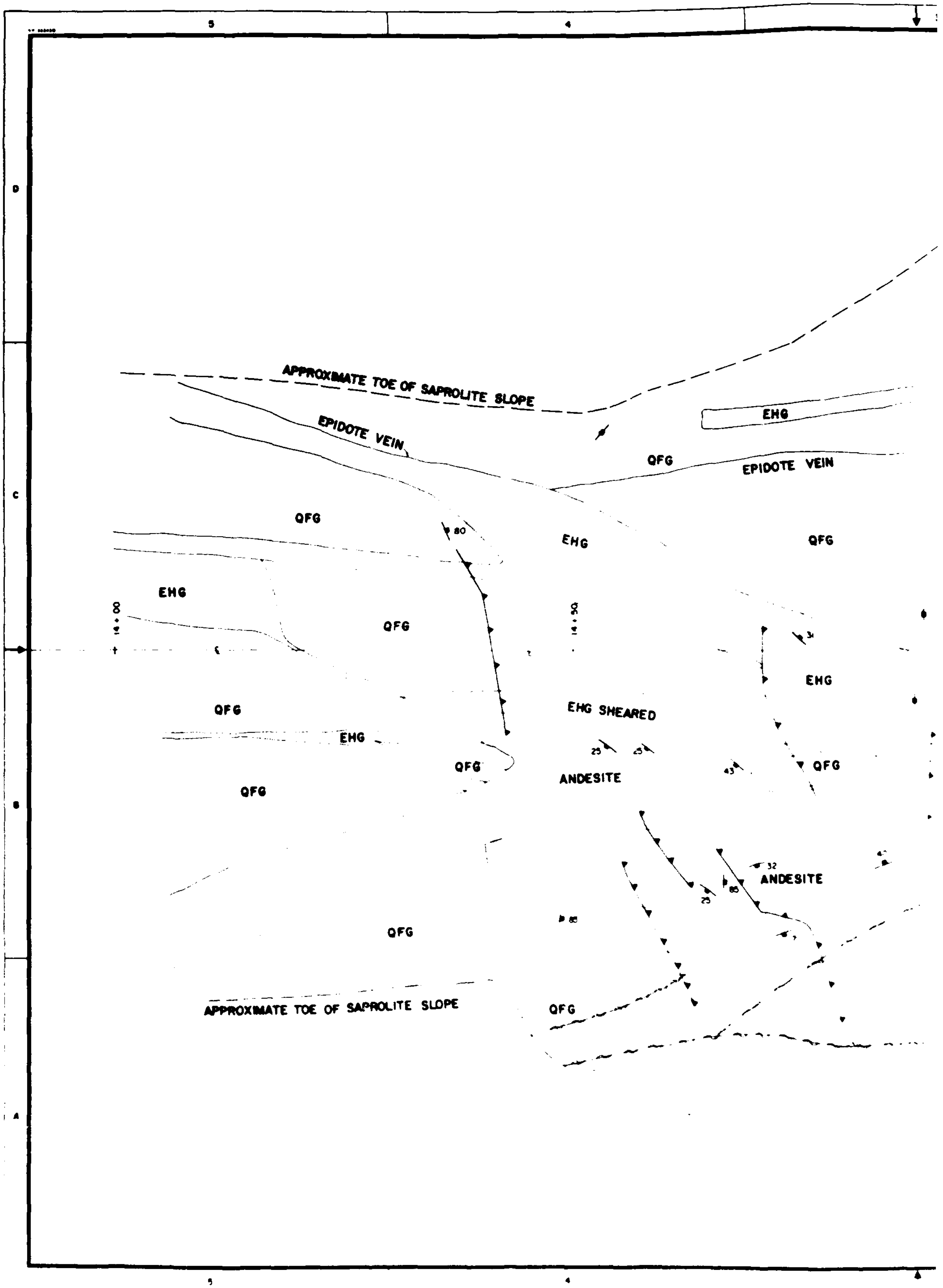
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
EMBANKMENTS
GEOLOGIC MAP-GEORGIA WEST CUTOFF
TRENCH-STA.12+00 TO 13+80
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

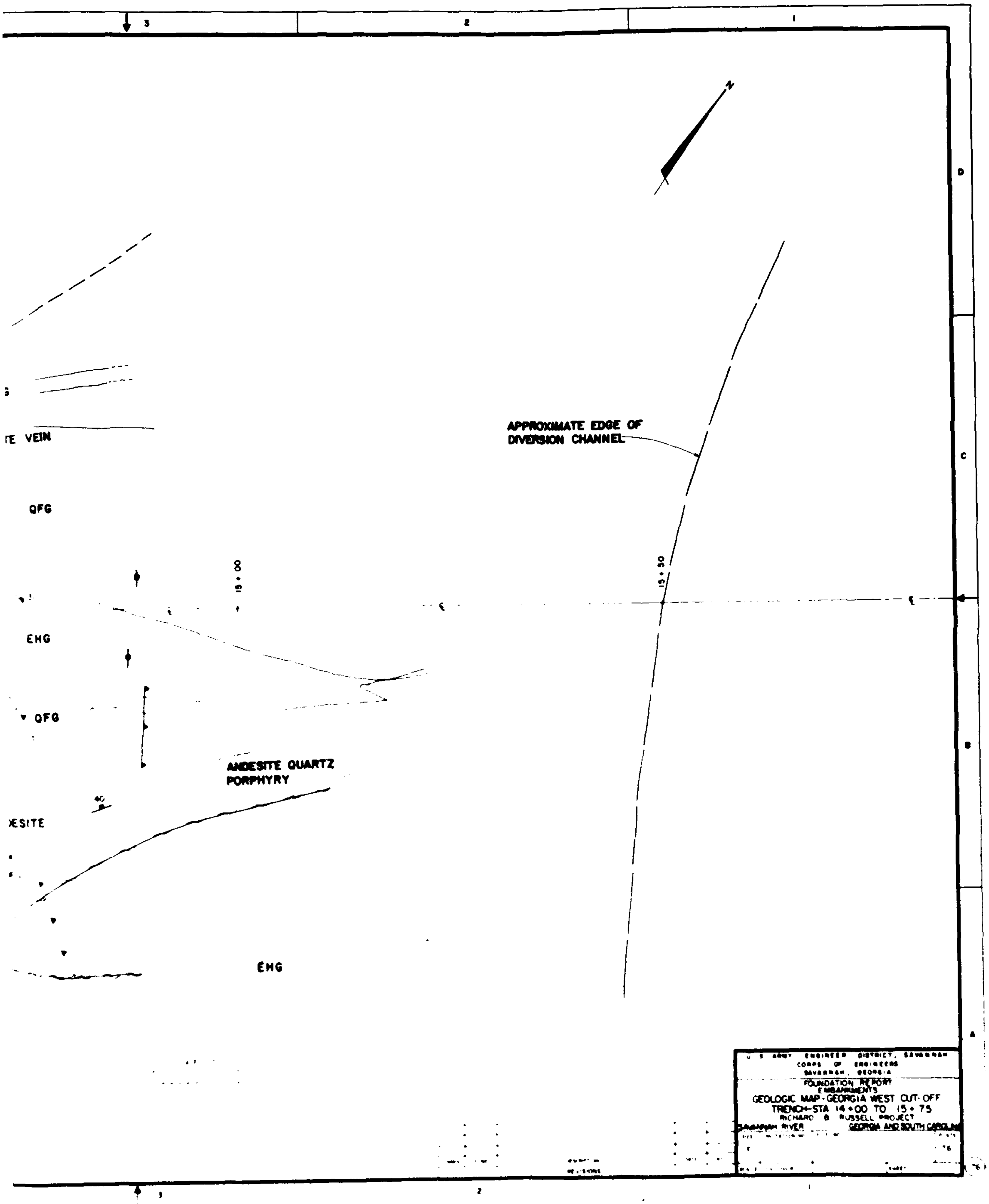
SIZE			PLATE
F			75

75

2

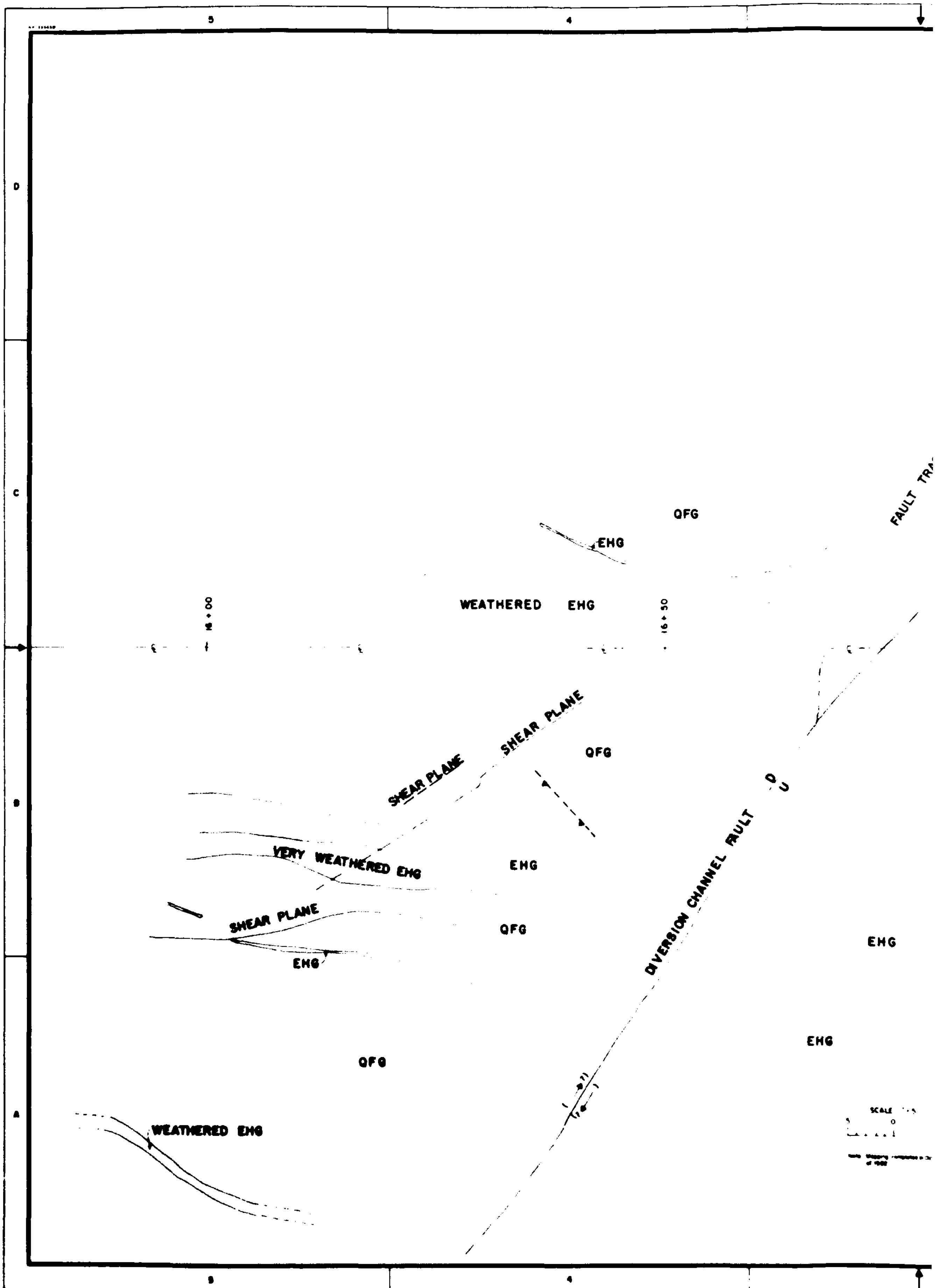


APPROXIMATE TOE OF SAPROLITE SLOPE
EPIDOTE VEIN
EHG
QFG
EPIDOTE VEIN
QFG
EHG
QFG
EHG
QFG
14+00
14+50
EHG
QFG
EHG SHEARED
EHG
QFG
QFG
ANDESITE
QFG
QFG
ANDESITE
QFG
APPROXIMATE TOE OF SAPROLITE SLOPE
QFG
5
4
D
C
B
A

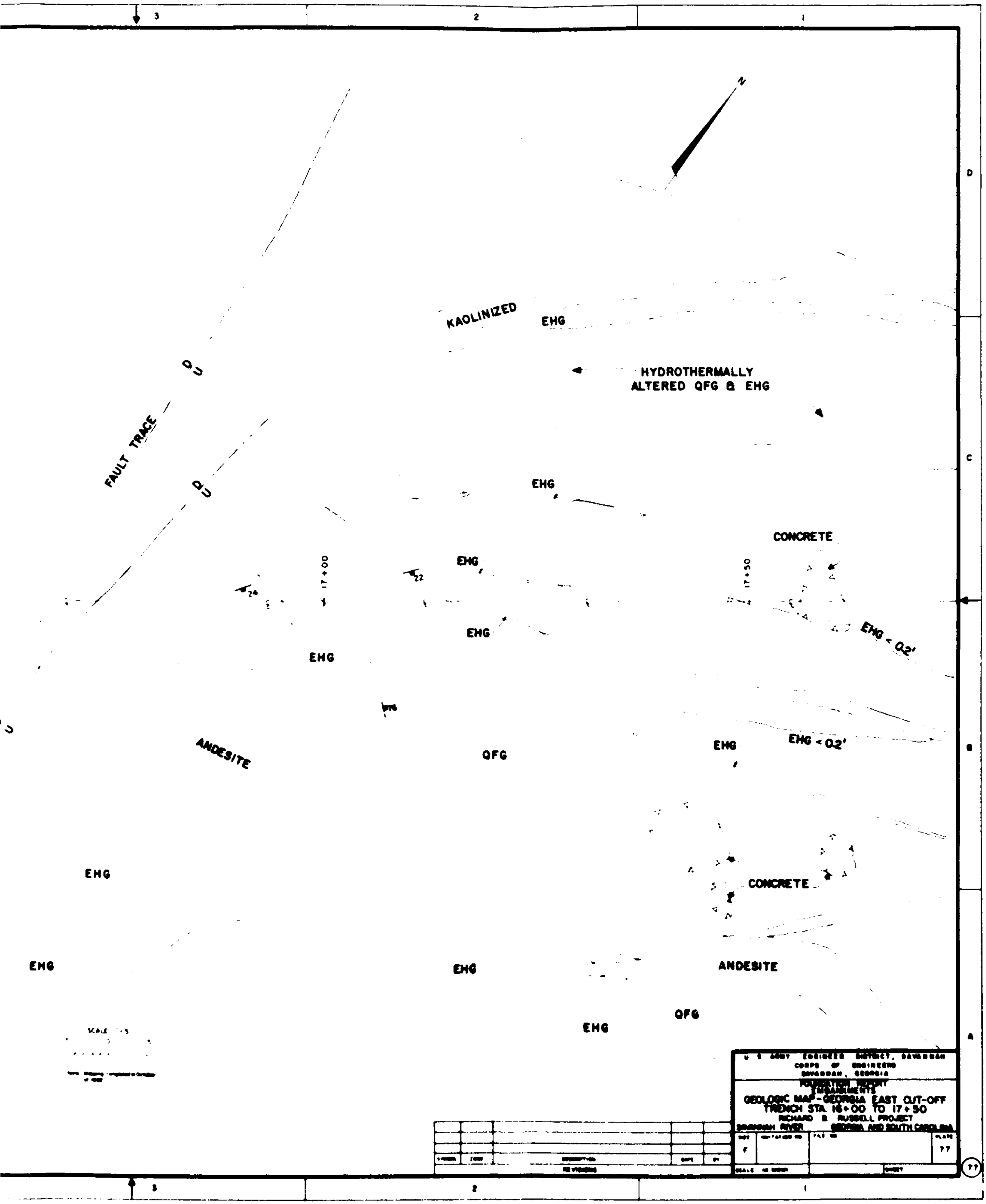


3
2
1
D
C
B
A
N
APPROXIMATE EDGE OF
DIVERSION CHANNEL
TE VEIN
QFG
15 + 00
15 + 50
EHG
QFG
ANDESITE QUARTZ
PORPHYRY
40
ESITE
EHG
REVISIONS
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
GEOLOGIC MAP - GEORGIA WEST CUT-OFF
TRENCH-STA 14 + 00 TO 15 + 75
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER
GEORGIA AND SOUTH CAROLINA

2

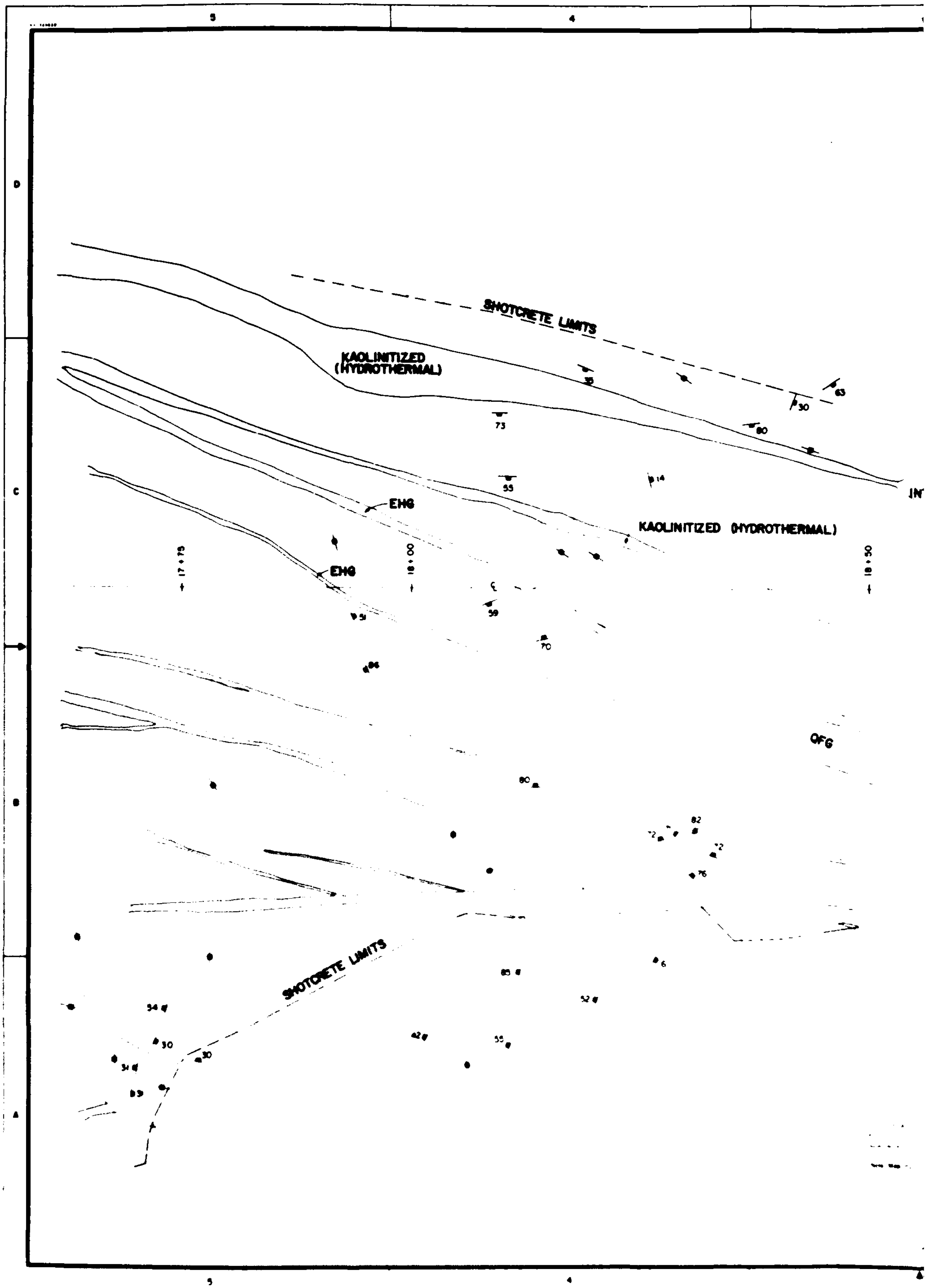


5
4
D
C
B
A
QFG
EHG
WEATHERED EHG
16+00
16+50
SHEAR PLANE
SHEAR PLANE
SHEAR PLANE
QFG
VERY WEATHERED EHG
EHG
SHEAR PLANE
QFG
EHG
DIVERSION CHANNEL FAULT
D
U
EHG
EHG
QFG
WEATHERED EHG
5
4

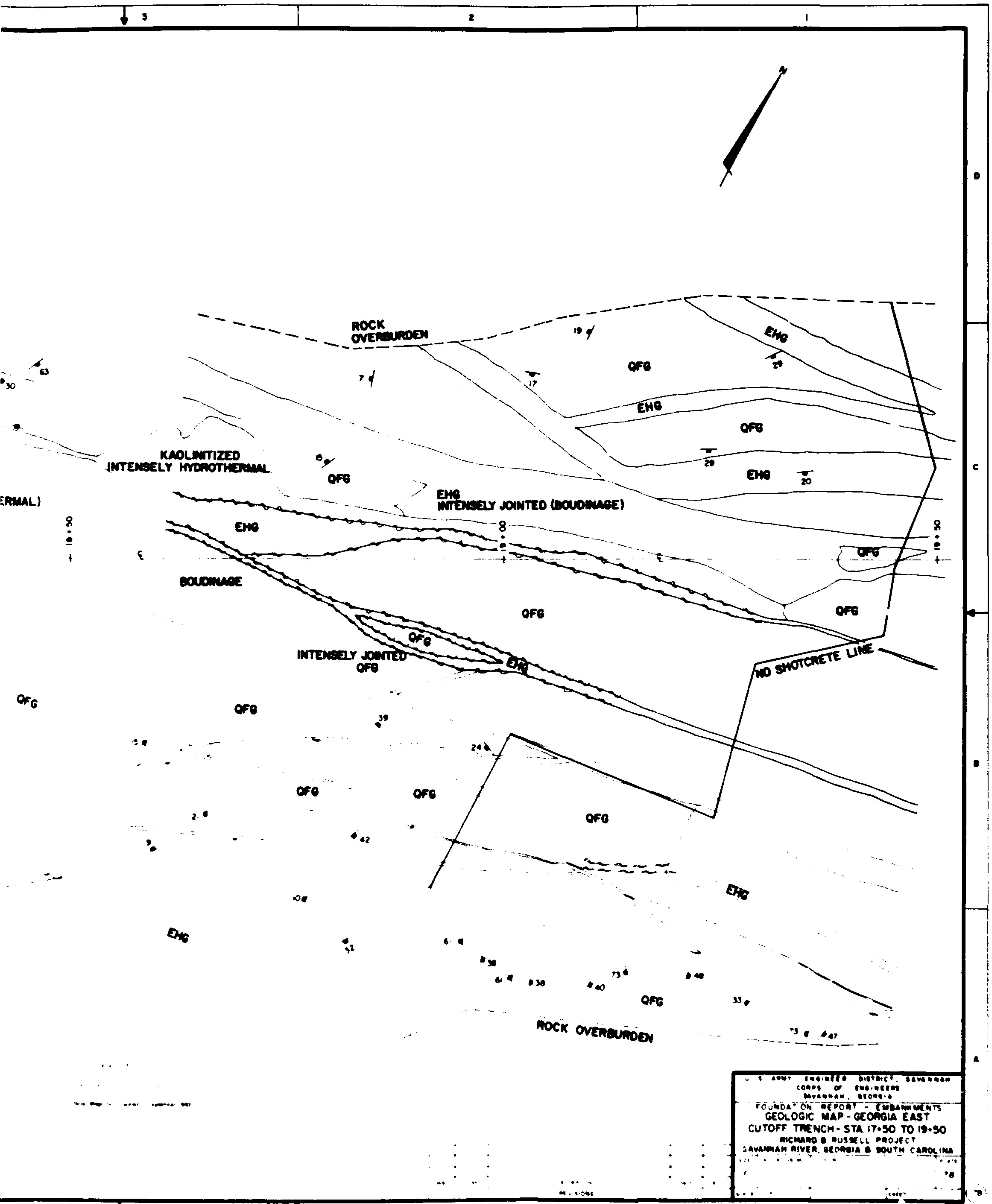


KAOLINIZED
EHG
HYDROTHERMALLY
ALTERED QFG & EHG
FAULT TRACE
D
U
EHG
CONCRETE
17+00
17+50
22
24
EHG
EHG
EHG < 0.2'
EHG
ANDESITE
QFG
EHG
EHG < 0.2'
EHG
CONCRETE
EHG
EHG
ANDESITE
QFG
EHG
GEOLOGIC MAP - GEORGIA EAST CUT-OFF
TRENCH STA. 16+00 TO 17+50
RICHARD B. RUSSELL PROJECT
77

2

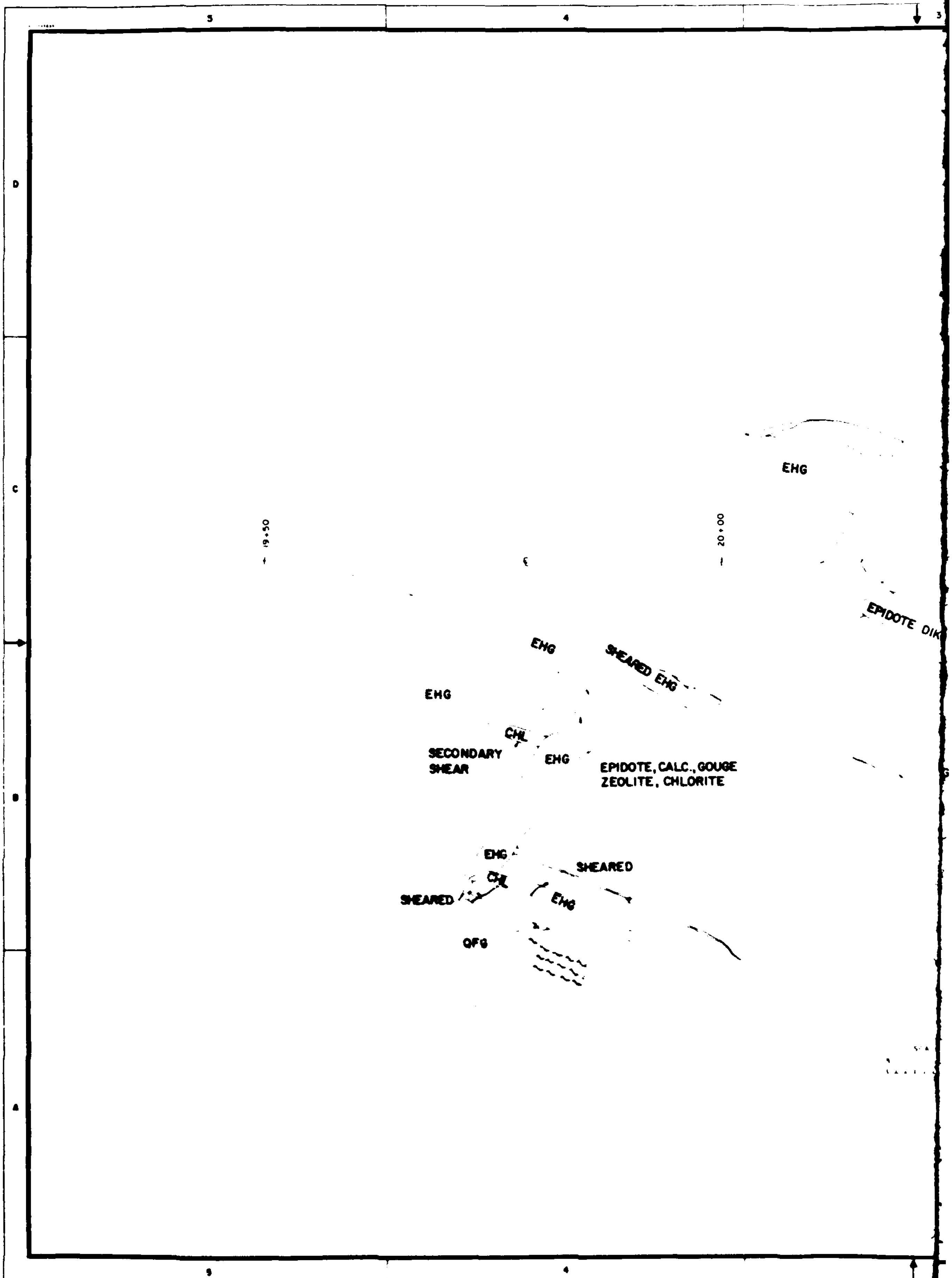


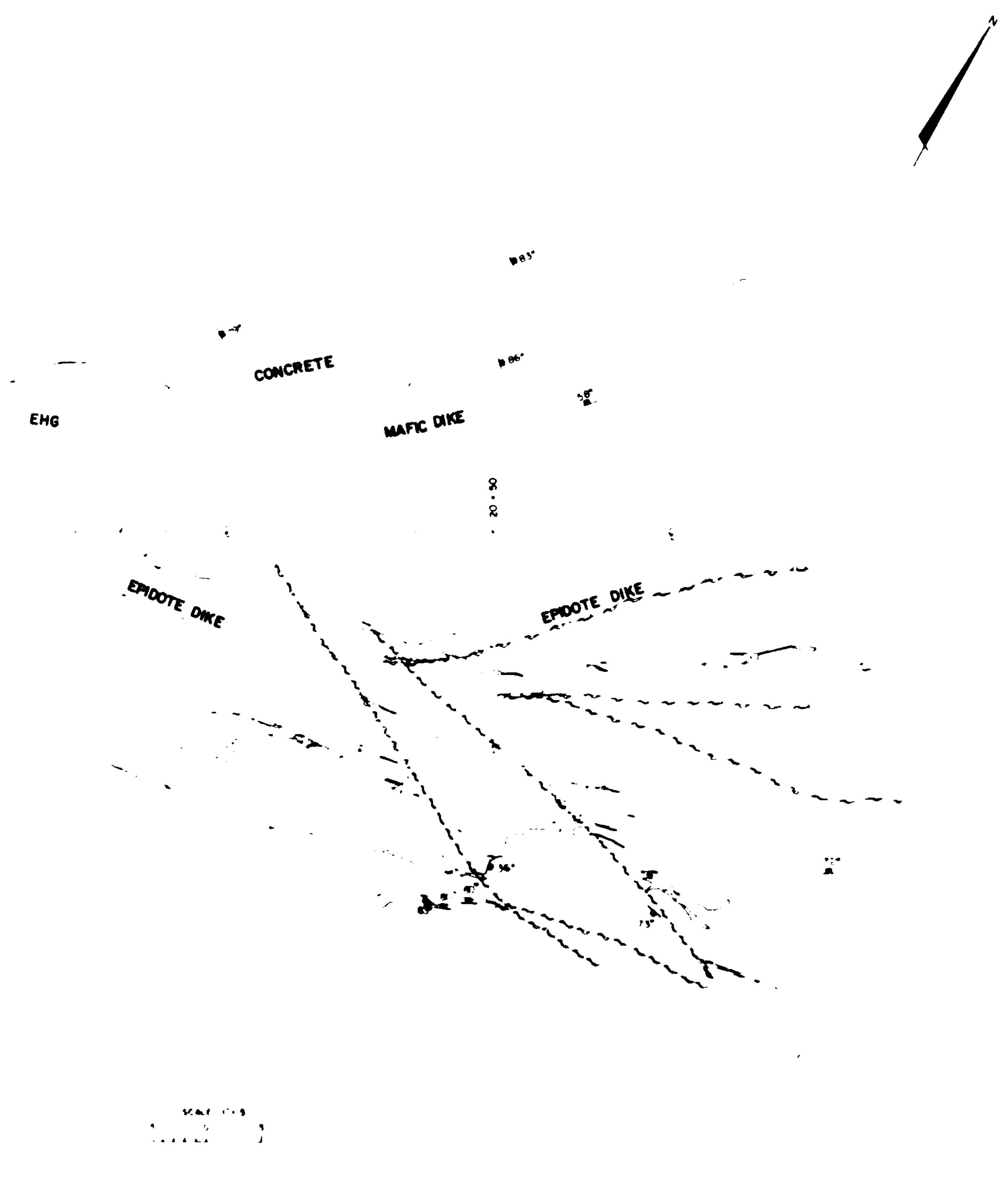
5
4
D
C
B
A
SHOTCRETE LIMITS
KAOLINITIZED
(HYDROTHERMAL)
KAOLINITIZED (HYDROTHERMAL)
EHG
EHG
17+75
18+00
18+50
QFG
SHOTCRETE LIMITS



U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT - EMBANKMENTS
GEOLOGIC MAP - GEORGIA EAST
CUTOFF TRENCH - STA 17+50 TO 19+50
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER, GEORGIA & SOUTH CAROLINA


19+50
20+00
EHG
EPIDOTE DIK
EHG
SHEARED EHG
EHG
CHL
SECONDARY
SHEAR
EHG
EPIDOTE, CALC., GOUGE
ZEOLITE, CHLORITE
EHG
SHEARED
CHL
SHEARED
EHG
QFG

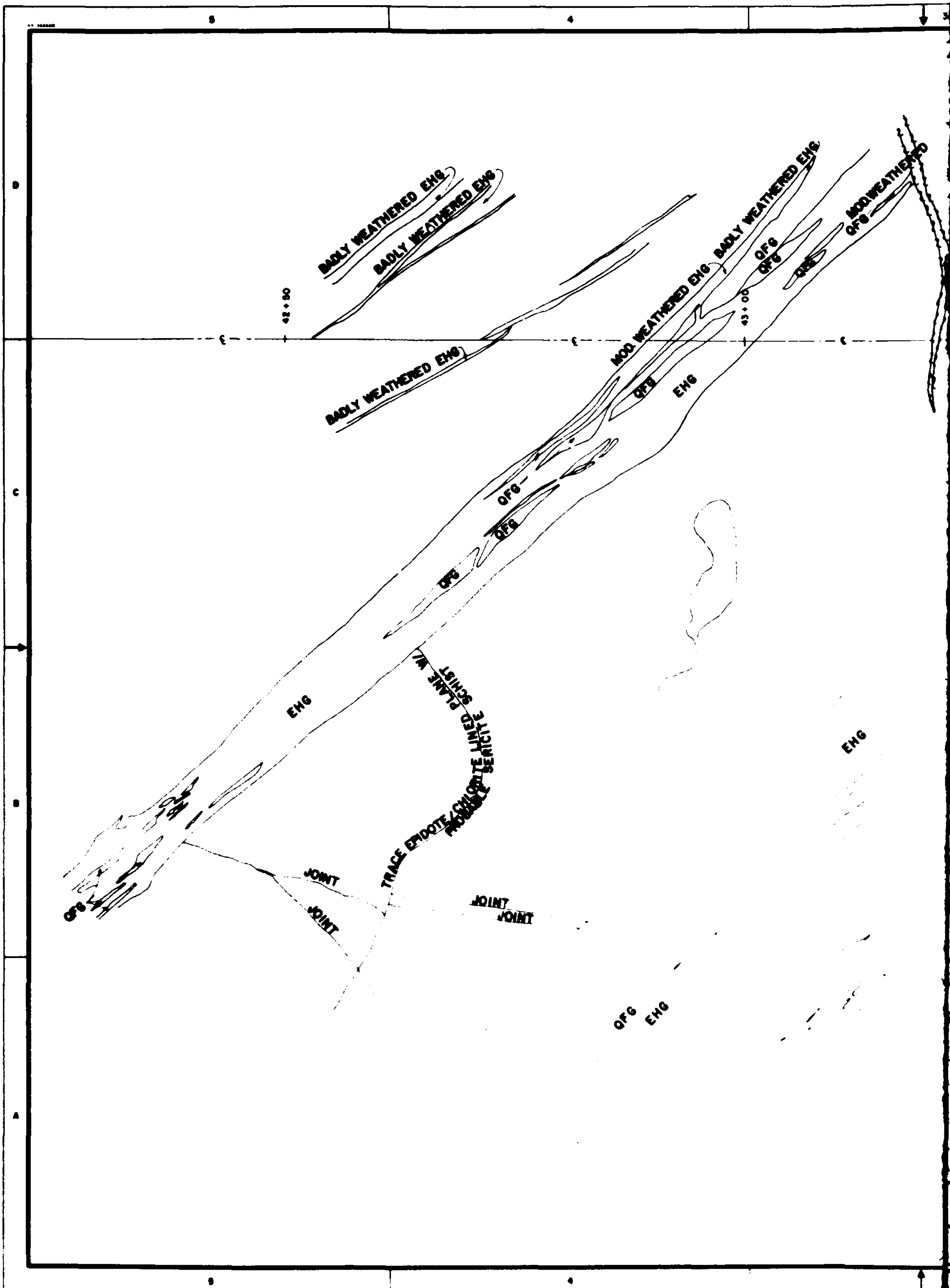


U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

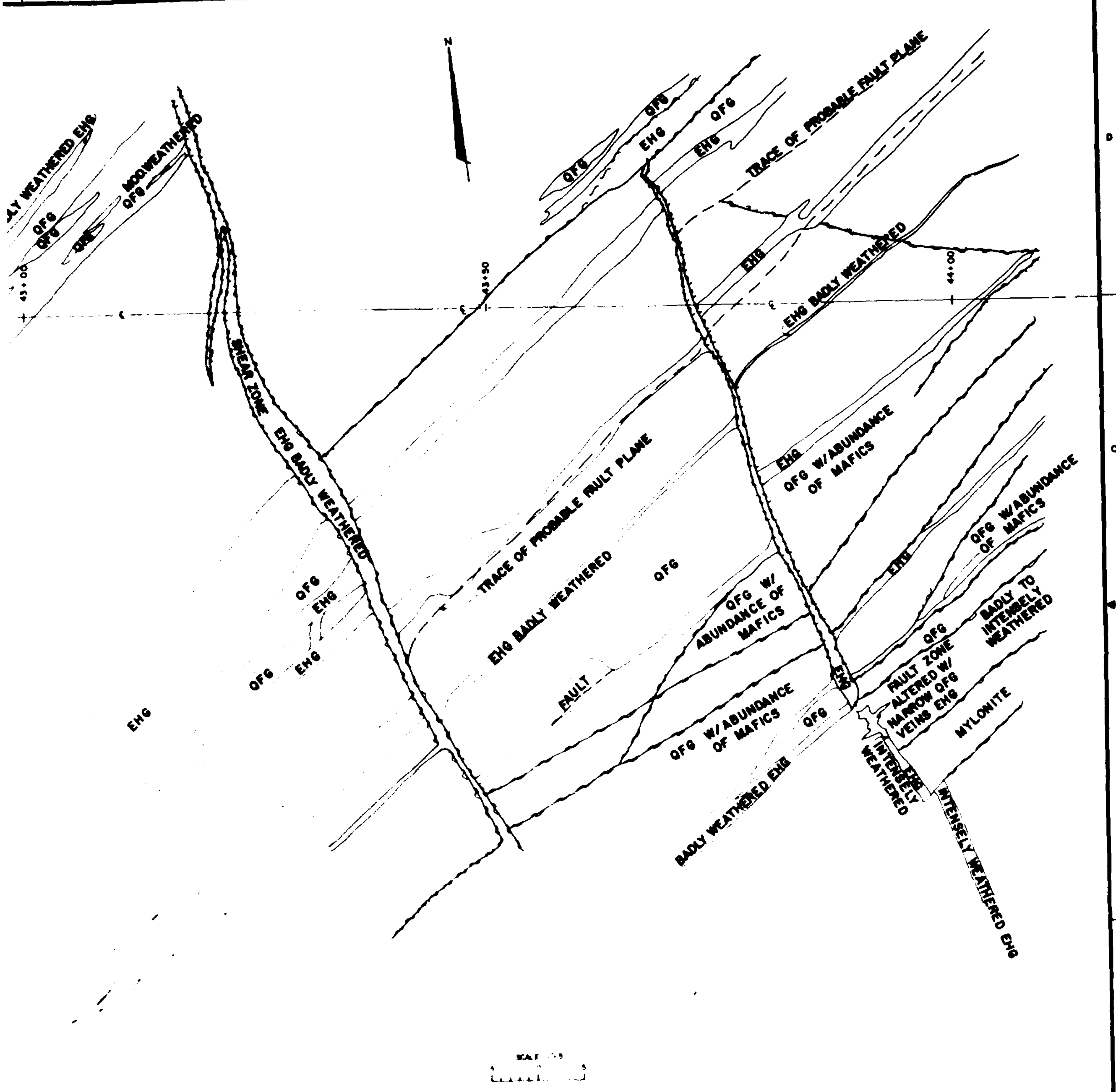
FOUNDATION REPORT
EMBANKMENTS
GEOLOGIC MAP-GEORGIA EAST CUT-OFF
TRENCH-STA 19+00 TO 20+80
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 79

2



5
4
D
C
B
A
BADLY WEATHERED EHG
BADLY WEATHERED EHG
BADLY WEATHERED EHG
BADLY WEATHERED EHG
MOD. WEATHERED EHG
MOD. WEATHERED EHG
MOD. WEATHERED
42+50
43+00
QFG
EHG
TRACE EPIDOTE/CHLORITE LINED PLANE W/ PROBABLE SERICITE SCHIST
JOINT
JOINT
JOINT
JOINT

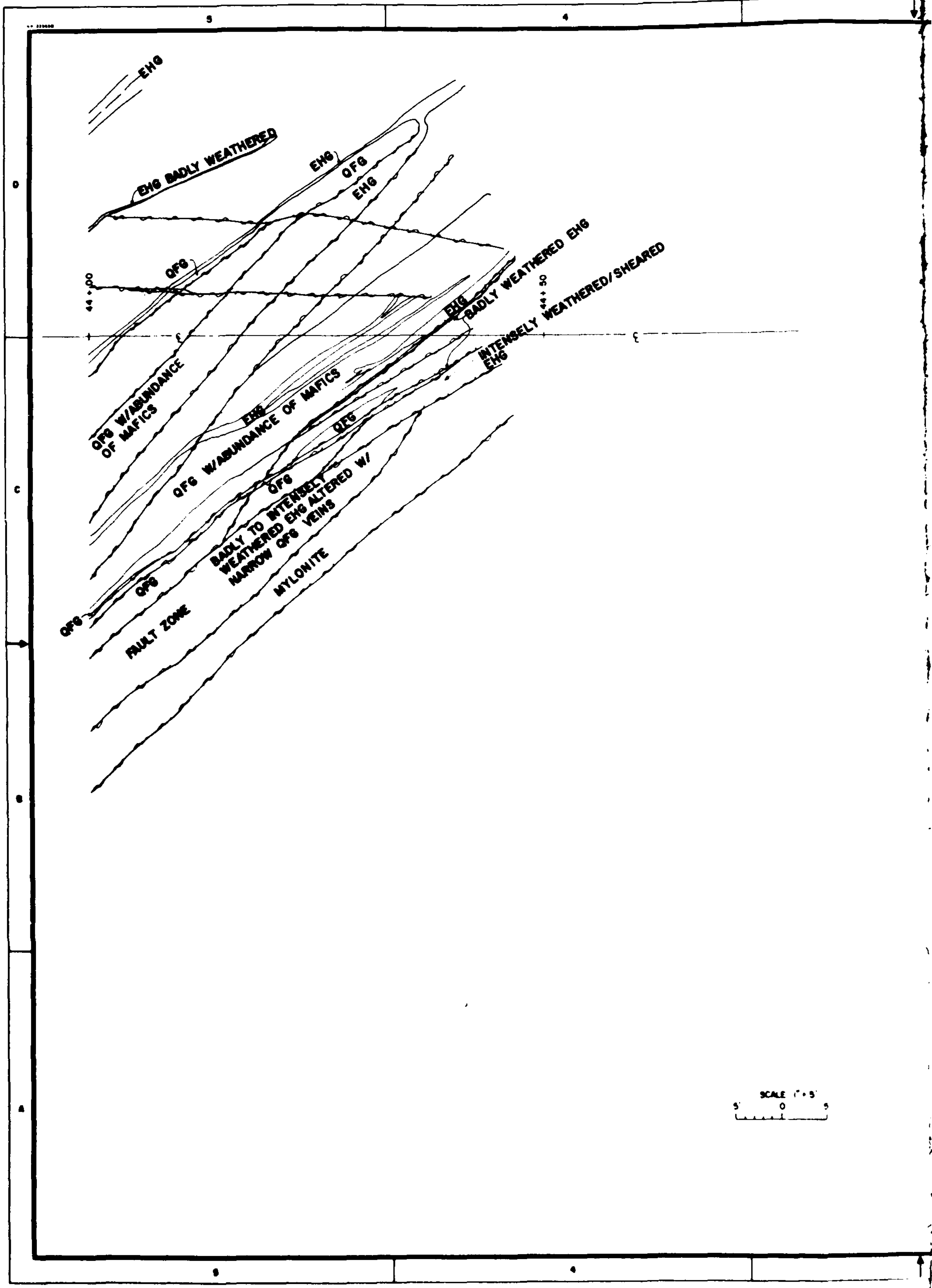


U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

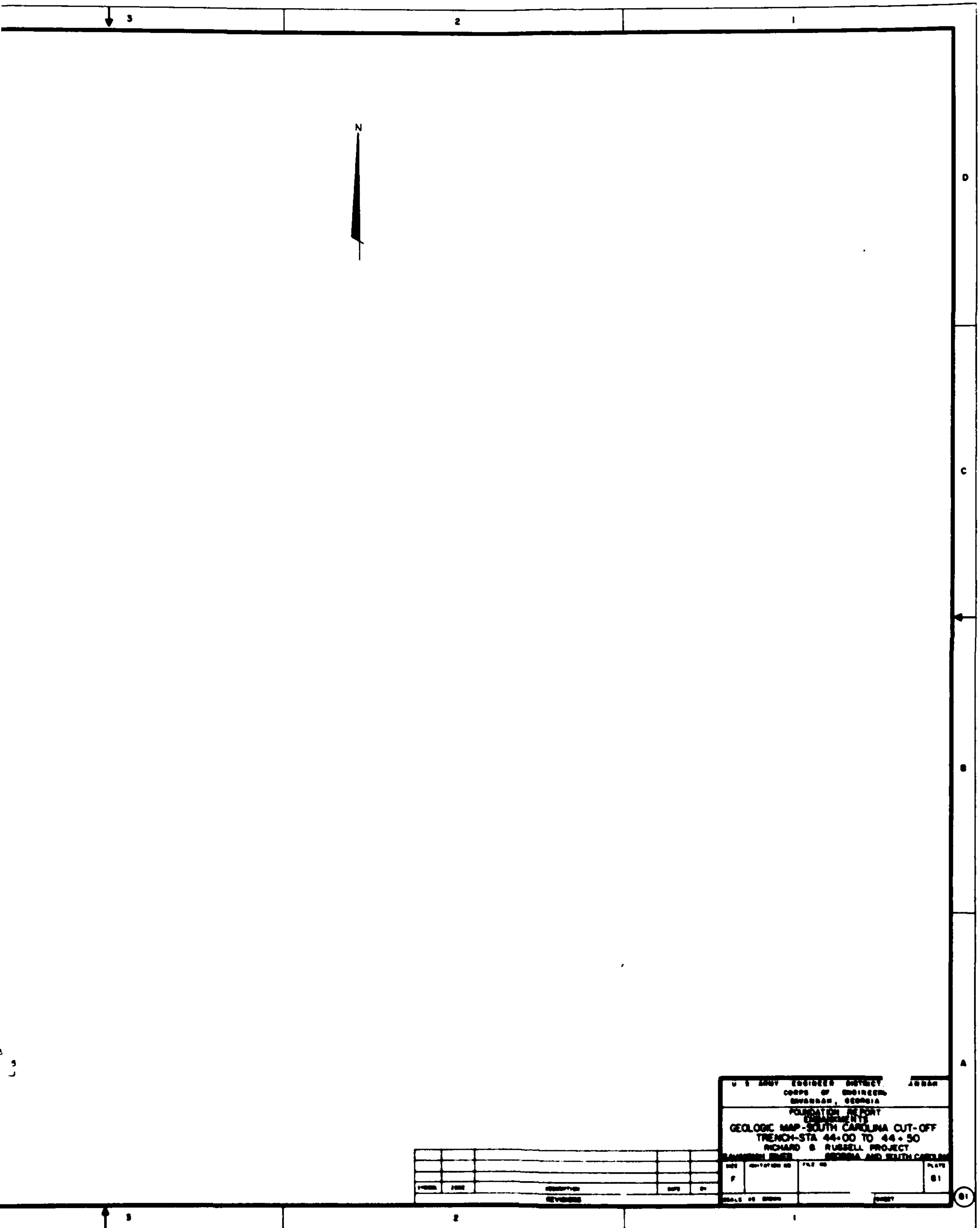
FOUNDATION REPORT
EMBANKMENTS
GEOLOGIC MAP - SOUTH CAROLINA CUT-OFF
TRENCH-STA 42+25 TO 44+00
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 80

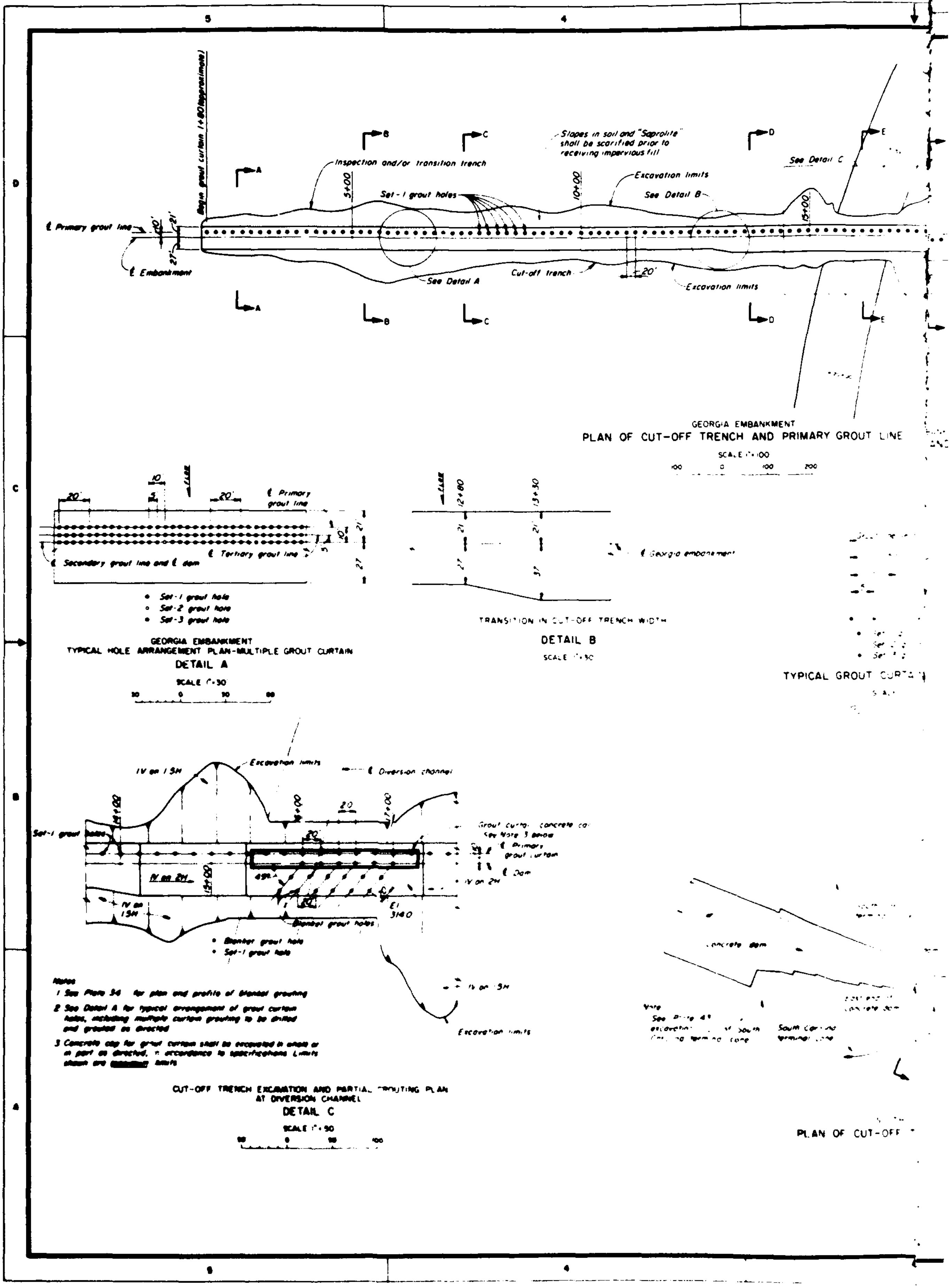
2



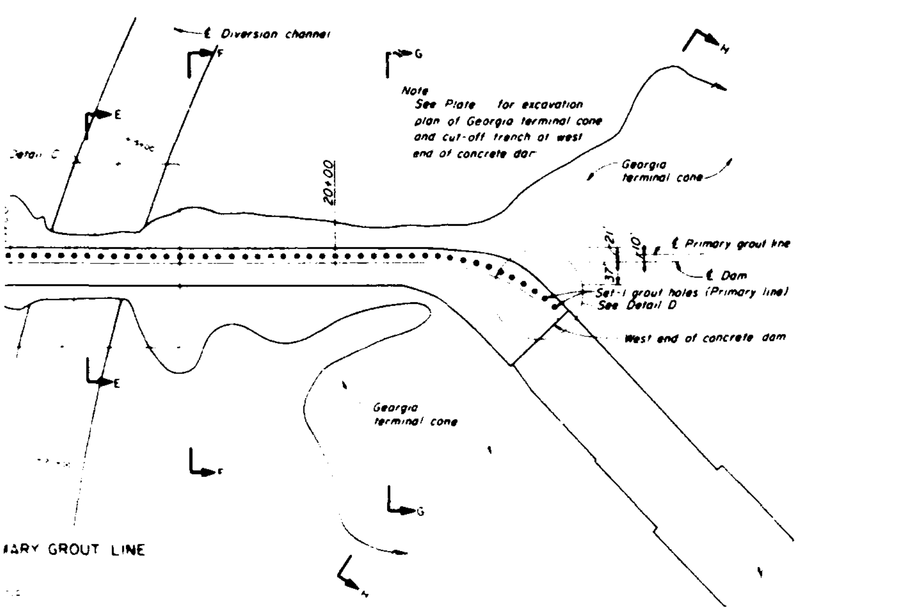
5
4
D
C
B
A
EHG
EHG BADLY WEATHERED
EHG
QFG
EHG
QFG
44+00
44+50
EHG
BADLY WEATHERED EHG
INTENSELY WEATHERED/SHEARED
EHG
QFG W/ABUNDANCE OF MAFICS
EHG
QFG W/ABUNDANCE OF MAFICS
QFG
QFG
BADLY TO INTENSELY WEATHERED EHG ALTERED W/ NARROW QFG VEINS
QFG
QFG
FAULT ZONE
MYLONITE
SCALE 1" = 5'
5 0 5


3
2
1
N
D
C
B
A
U S ARMY ENGINEER DISTRICT ANNAN
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
GEOLOGIC MAP-SOUTH CAROLINA CUT-OFF
TRENCH-STA 44+00 TO 44+50
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
SIZE
F
FILE NO
PLATE
81
SCALE AS SHOWN
SHEET
REVISIONS
81

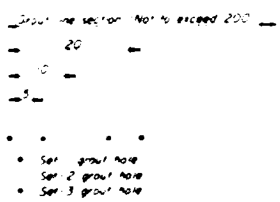
2



Begin grout curtain 1+80 (approximate)
℄ Primary grout line
℄ Embankment
Inspection and/or transition trench
Set-1 grout holes
Slopes in soil and "Saprolite" shall be scarified prior to receiving impervious fill
Excavation limits
See Detail B
See Detail C
See Detail A
Cut-off trench
5+00
10+00
15+00
GEORGIA EMBANKMENT
PLAN OF CUT-OFF TRENCH AND PRIMARY GROUT LINE
℄ Primary grout line
Secondary grout line and ℄ dam
℄ Tertiary grout line
Set-1 grout hole
Set-2 grout hole
Set-3 grout hole
GEORGIA EMBANKMENT
TYPICAL HOLE ARRANGEMENT PLAN-MULTIPLE GROUT CURTAIN
DETAIL A
12+80
13+30
℄ Georgia embankment
TRANSITION IN CUT-OFF TRENCH WIDTH
DETAIL B
TYPICAL GROUT CURTAIN
Excavation limits
℄ Diversion channel
1V on 1.5H
1V on 2H
Set-1 grout holes
Grout curtain concrete cap See Note 3 below
℄ Primary grout curtain
℄ Dam
Blanket grout holes
El 314.0
Blanket grout hole
Set-1 grout hole
Concrete dam
South Carolina terminal cone
Notes
1 See Plate 54 for plan and profile of blanket grouting
2 See Detail A for typical arrangement of grout curtain holes, including multiple curtain grouting to be drilled and grouted as directed
3 Concrete cap for grout curtain shall be excavated in whole or in part as directed, in accordance to specifications. Limits shown are minimum limits
CUT-OFF TRENCH EXCAVATION AND PARTIAL GROUTING PLAN AT DIVERSION CHANNEL
DETAIL C
PLAN OF CUT-OFF

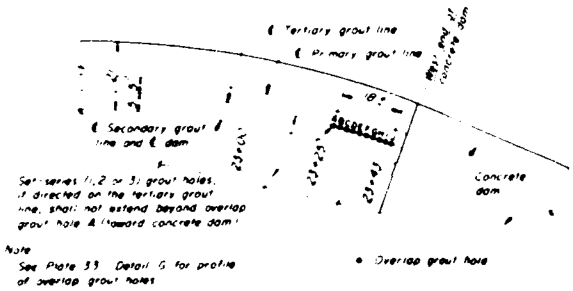


ARY GROUT LINE



AL GROUT CURTAIN HOLE SPACING

SCALE 1"=10

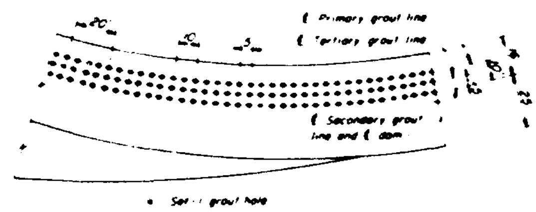


GEORGIA EMBANKMENT
OVERLAP GROUT HOLES BETWEEN EARTH EMBANKMENT AND CONCRETE DAM GROUT CURTAINS

DETAIL D

SCALE 1"=20

20 0 20 40

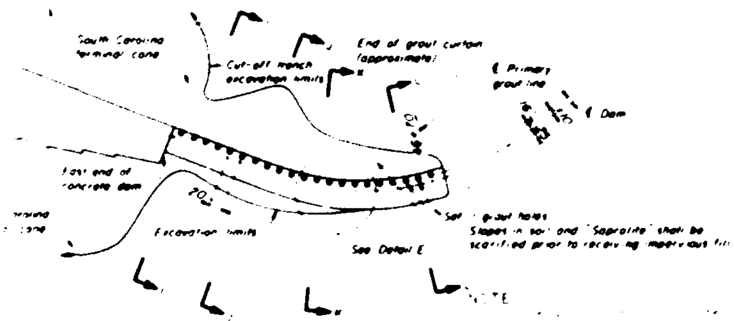


SOUTH CAROLINA EMBANKMENT
TYPICAL HOLE ARRANGEMENT PLAN-MULTIPLE GROUT CURTAIN

DETAIL E

SCALE 1"=30'

30 0 30



SOUTH CAROLINA EMBANKMENT
LAN OF CUT-OFF TRENCH AND PRIMARY GROUT LINE

SCALE 1"=100

NOTES

1. SEE PLATE 33 FOR PROFILE ALONG GROUT CURTAIN.
2. SEE PLATES 27 THRU 31 FOR CROSS-SECTIONS A-A THRU L-L.

GENERAL GROUTING NOTES

1. SEE PLATES 10 AND 11 FOR PLAN OF BORINGS DRILLED PREVIOUSLY.
2. SEE SPECIFICATIONS APPENDICES FOR LOGS OF PREVIOUS BORINGS AND PRESSURE TEST DATA SHEETS.
3. SPACING, ANGLE, AND DIRECTION OF THE GROUT HOLES SHOWN ON THE DRAWINGS ARE TENTATIVE AND MAY BE ADJUSTED IN THE FIELD BY THE CONTRACTING OFFICER.
4. ALL CURTAIN GROUT HOLES SHALL BE GROUTED BY THE STAGE-GROUTING METHOD; BLANKET (CONSOLIDATION) GROUT HOLES SHALL BE GROUTED BY THE STOP-GROUTING METHOD, IN ACCORDANCE TO SECTION: "FOUNDATION DRILLING AND GROUTING" OF THE SPECIFICATIONS.
5. ACTUAL DEPTH, ORDER OF DRILLING AND GROUTING, AND GROUTING PRESSURE, WILL BE DETERMINED IN THE FIELD AS THE GROUTING PROGRAM PROGRESSES, AND WILL BE AS DIRECTED BY THE CONTRACTING OFFICER.
6. EXPLORATORY CORE BORINGS SHALL BE DRILLED, AS DIRECTED, TO DETERMINE THE CONDITION OF THE ROCK PRIOR TO GROUTING AND/OR THE EFFECTIVENESS OF THE GROUTING OPERATIONS DURING AND AFTER GROUTING.
7. TYPICAL GROUT CURTAIN(S) AND/OR BLANKET GROUTING GRID SHOWN REPRESENT APPROXIMATE SPACING OF HOLES. ADDITIONAL HOLES SHALL BE PROVIDED BY USING THE SPLIT-SPACING METHOD, AS DIRECTED.
8. GROUT CURTAIN(S) SHALL BE LAID OUT IN SECTIONS, WHICH ARE REACHES ALONG THE CURTAIN THAT ARE NO GREATER THAN 200 FEET IN LENGTH AND IN WHICH GROUTING OPERATIONS WILL NOT BE PERMITTED AT THE SAME TIME DRILLING IS IN PROGRESS. INSOFAR AS IS PRACTICABLE THE GROUT CURTAIN(S) WILL BE SUBDIVIDED INTO SECTIONS IN A MANNER WHICH WILL FACILITATE THE CONTRACTOR'S OPERATIONS.
9. MULTIPLE CURTAIN GROUT LINES (SHOWN TYPICALLY IN DETAILS A AND E) SHALL BE DRILLED AND GROUTED IN ANY SECTION, AS DIRECTED, IN ACCORDANCE TO PROCEDURES IN SPECIFICATIONS.
10. CURTAIN GROUTING MAY REQUIRE ANY NUMBER OF STAGES PER ZONE. GROUT CURTAIN HOLES SHALL BE INCLINED DOWNSTATION AS SHOWN ON TYPICAL SECTIONS, EXCEPT CURTAIN OVERLAP HOLES A THRU J, WHICH RANGE FROM VERTICAL TO VARYING ANGLES UPSTATION, AS SHOWN. BLANKET (CONSOLIDATION) GROUT HOLES IN THE FAULT-ZONE SHALL BE DRILLED VERTICAL (90 DEGREES FROM HORIZONTAL) UNLESS DIRECTED OTHERWISE.

ANY LATERAL OR LINEAR DEVIATION IN GROUT LINES AND/OR GRIDS FROM NORMALITY WITH THE AXIS OF THE DAM, SHALL BE DETERMINED BY THE CONTRACTING OFFICER TO MEET GEOLOGICAL CONDITIONS OF THE SITE. GROUT HOLES SHALL BE AT LEAST 1 1/2 INCHES IN DIAMETER AT THE POINT OF MAXIMUM PENETRATION.

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
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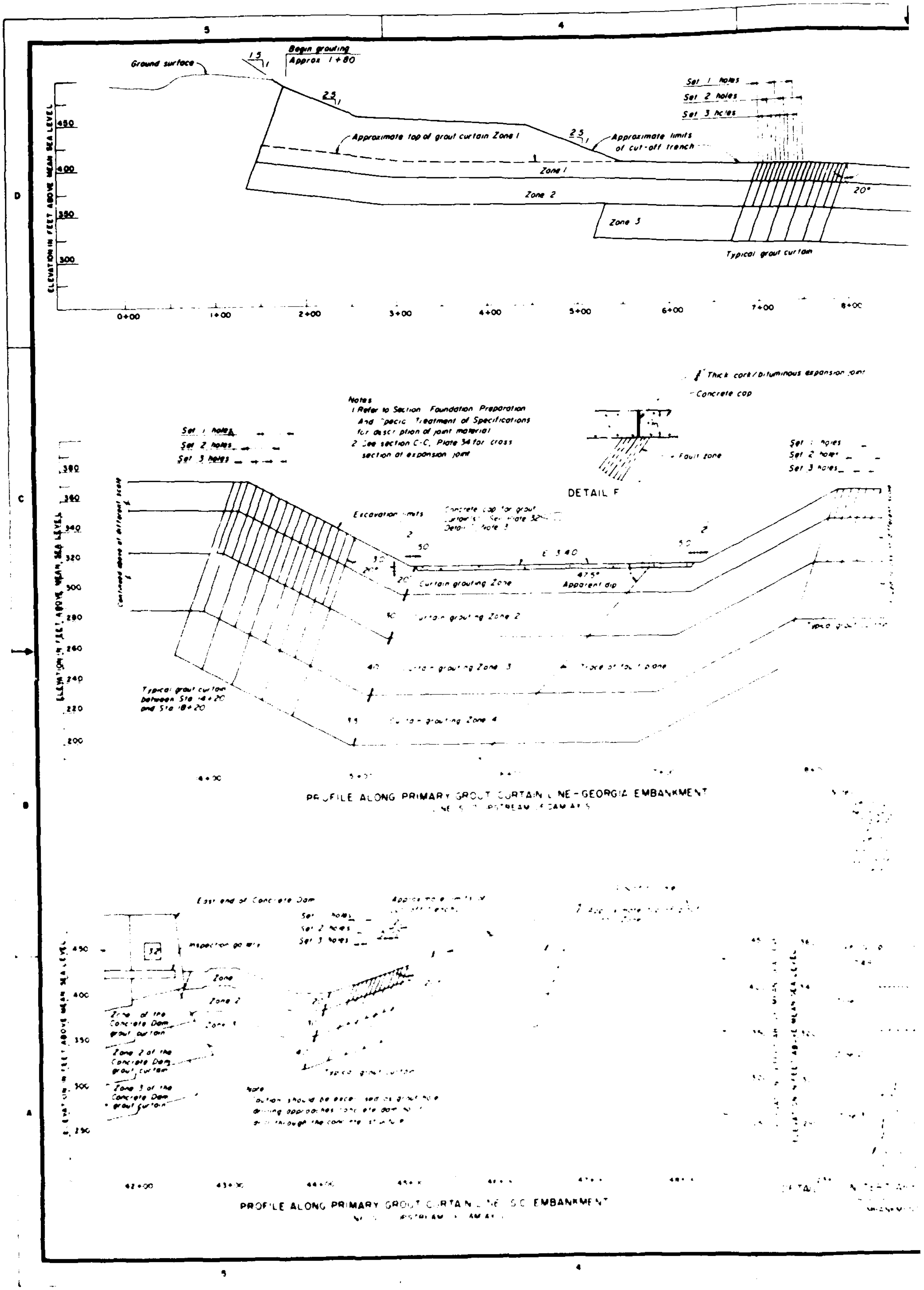
FOUNDATION REPORT
EMBANKMENTS
CUT-OFF TRENCH EXCAVATION AND GROUTING PLAN (AS-DESIGNED)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 82

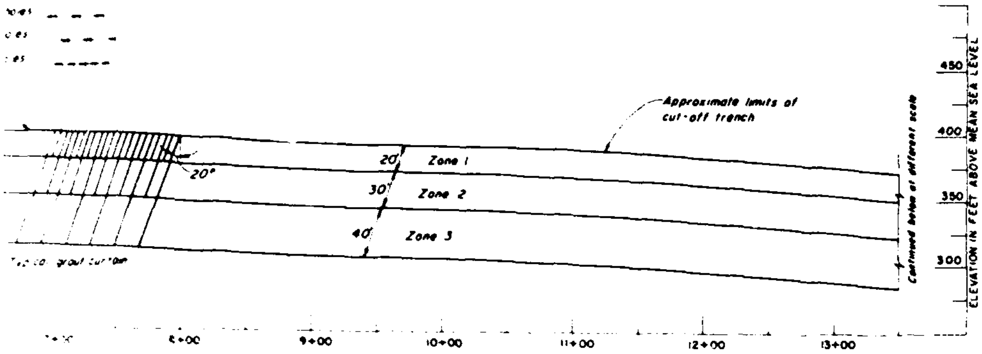
SCALE AS SHOWN

REVISIONS

2



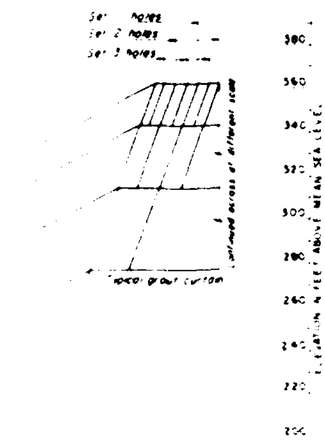
Ground surface
Begin grouting
Approx 1+80
Approximate top of grout curtain Zone 1
Approximate limits of cut-off trench
Set 1 holes
Set 2 holes
Set 3 holes
Zone 1
Zone 2
Zone 3
Typical grout curtain
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
0+00
1+00
2+00
3+00
4+00
5+00
6+00
7+00
Notes
1 Refer to Section Foundation Preparation
2 See section C-C, Plate 34 for cross section of expansion joint
Thick cork/bituminous expansion joint
Concrete cap
Fault zone
DETAIL F
Excavation limits
Curtain grouting Zone 1
Curtain grouting Zone 2
Curtain grouting Zone 3
Curtain grouting Zone 4
Apparent dip
Trace of fault plane
Typical grout curtain between Sta 14+20 and Sta 18+20
GEORGIA EMBANKMENT
East end of Concrete Dam
Inspection gallery
Zone 1 of the Concrete Dam grout curtain
Zone 2 of the Concrete Dam grout curtain
Zone 3 of the Concrete Dam grout curtain
Typical grout curtain
Note
42+00
43+00
EMBANKMENT

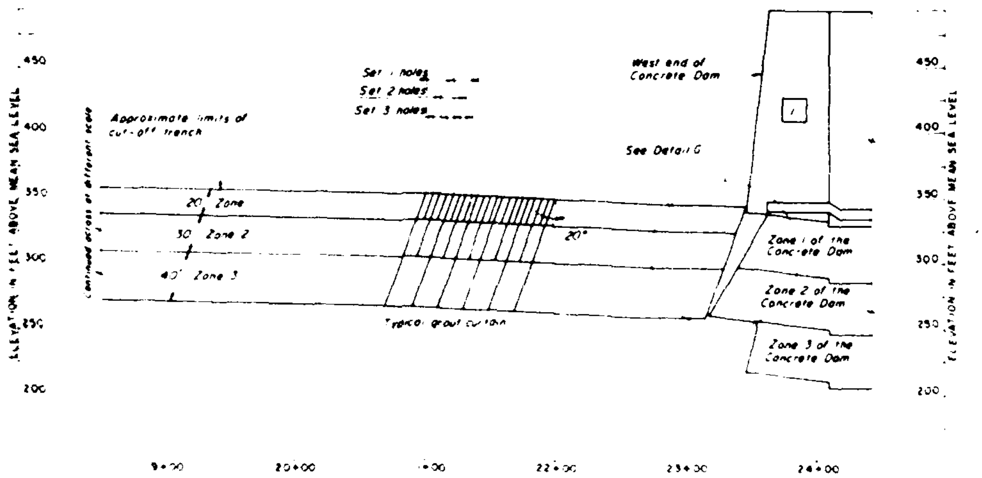


NOTE:

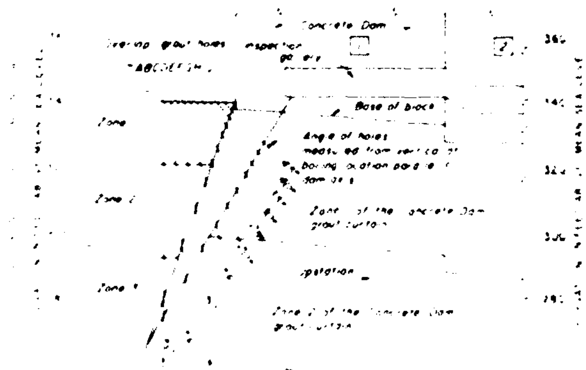
See Plate B2 for general grouting notes

NOTE

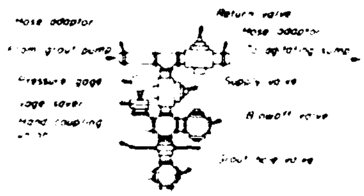




Notes



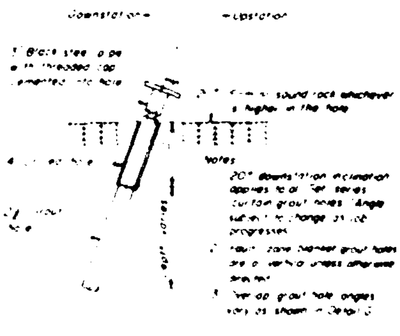
TERTIARY GROUT CURTAIN GEORGIA EMBANKMENT



GROUT MANIFOLD HEADER FOR CIRCULATING SYSTEM

A	[illegible]	[illegible]		[illegible]
B	[illegible]	[illegible] Upstation		[illegible]
	[illegible]	[illegible]		60
	[illegible]	[illegible]		50
	[illegible]	[illegible]		45
	[illegible]	[illegible]		4[illegible]
	[illegible]	[illegible]		35
H	[illegible]	[illegible]		[illegible]
	[illegible]	[illegible]		30
	[illegible]	[illegible]		25

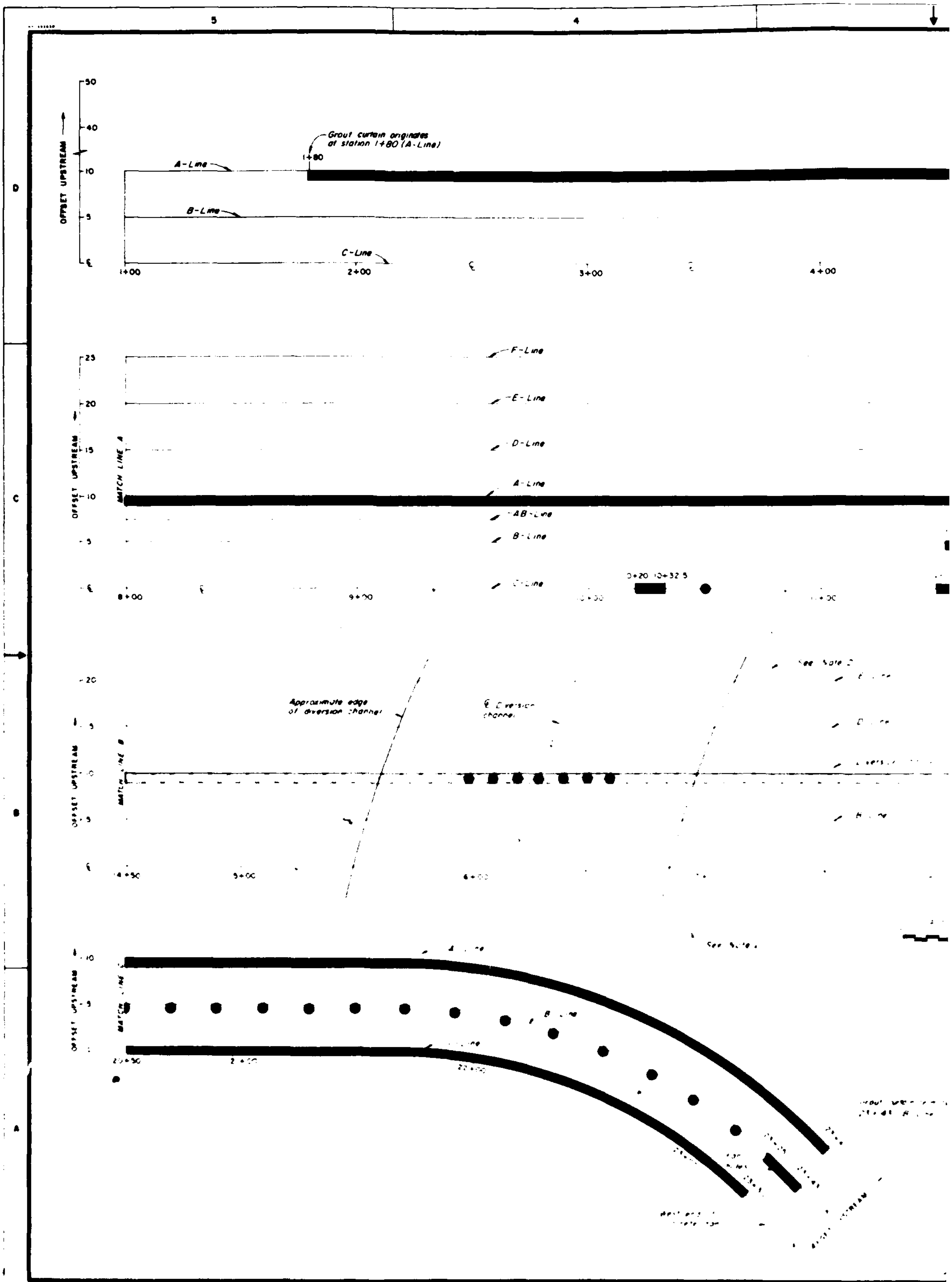
TABLE A



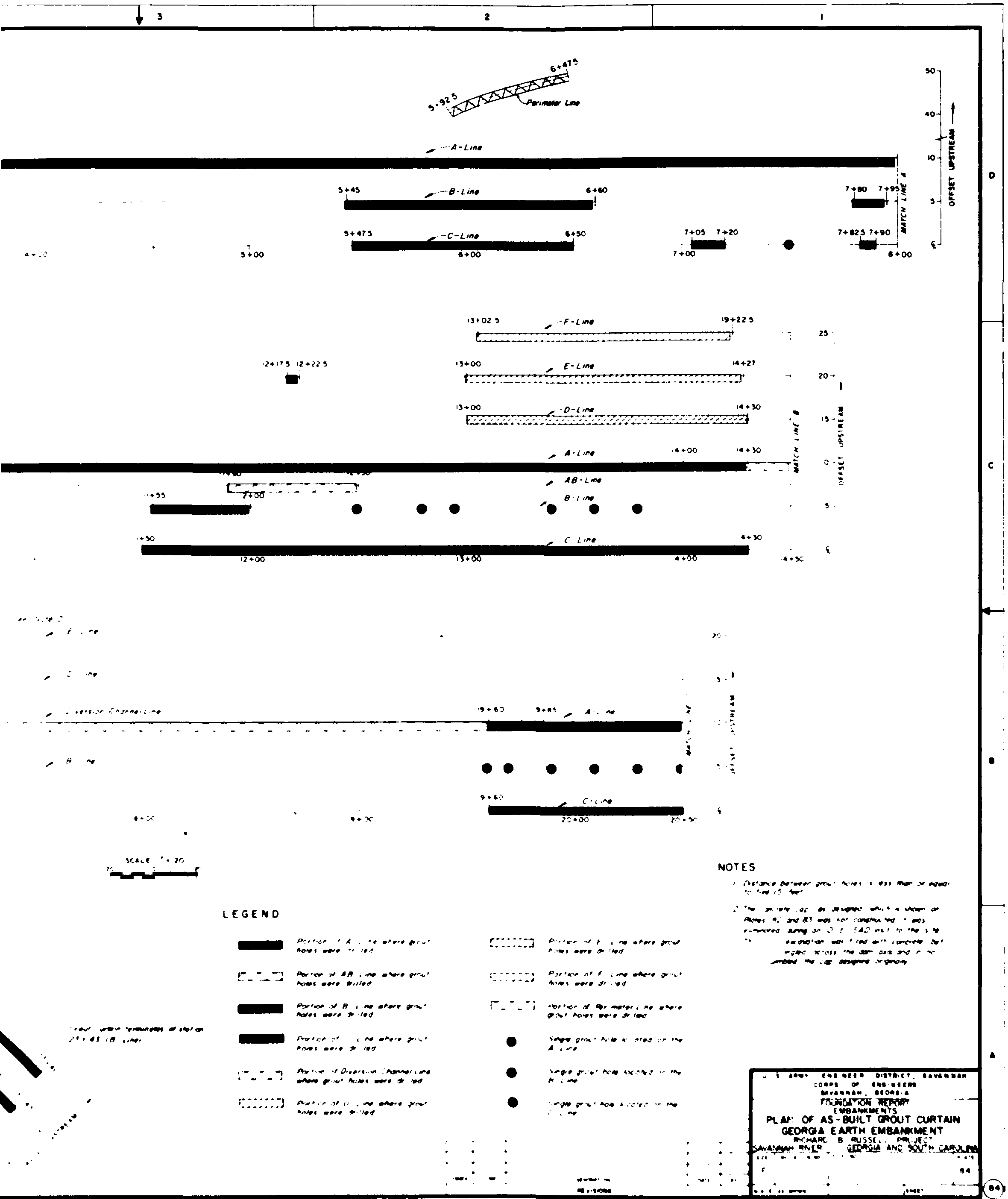
GROUT HOLE DETAIL

FOUNDATION REPORT
EMBANKMENTS
CURTAIN GROUTING
PROFILES (AS-DESIGNED)
RICHARD B RUSSELL PROJECT

B3

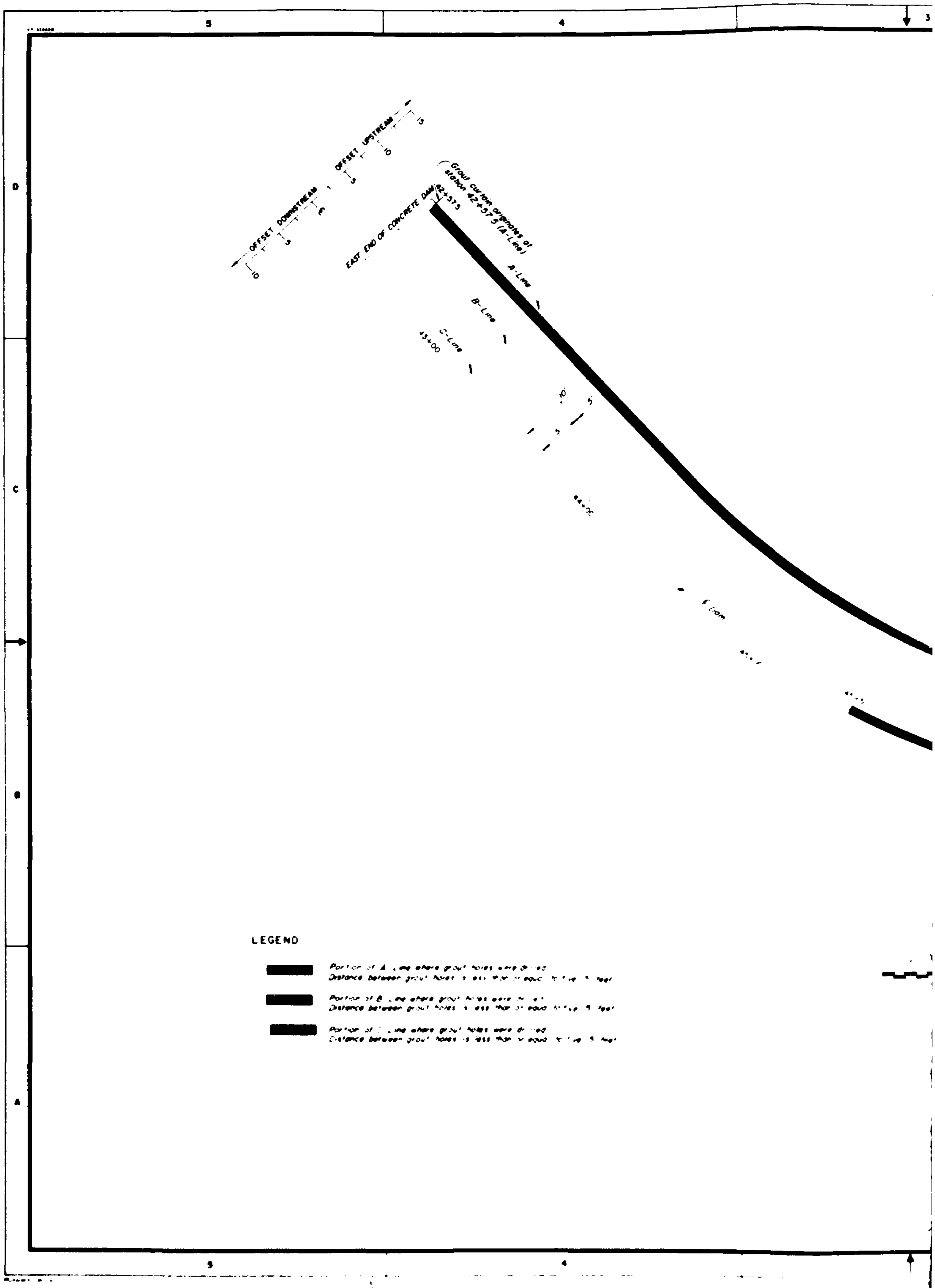


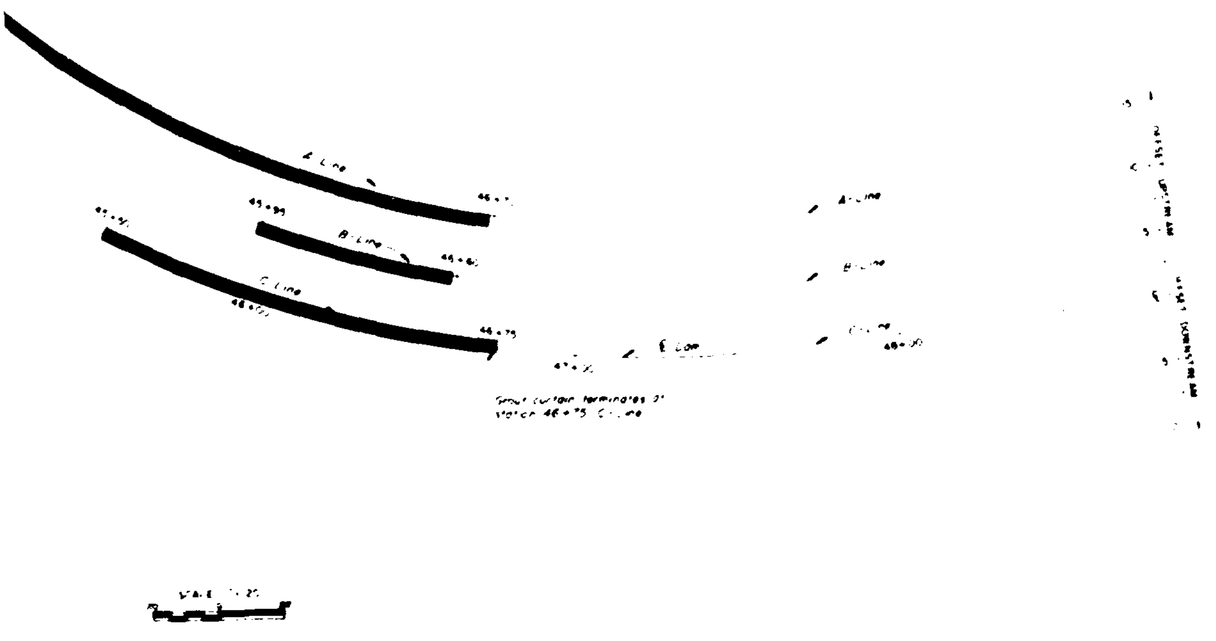
OFFSET UPSTREAM
Grout curtain originates at station 1+80 (A-Line)
1+80
A-Line
B-Line
C-Line
1+00
2+00
3+00
4+00
F-Line
E-Line
D-Line
A-Line
AB-Line
B-Line
MATCH LINE A
8+00
9+00
Approximate edge of diversion channel
MATCH LINE B



5+92.5
6+47.5
Perimeter Line
A-Line
B-Line
C-Line
5+45
6+60
5+47.5
6+50
5+00
6+00
7+05
7+20
7+00
7+80
7+95
7+82.5
7+90
8+00
MATCH LINE A
OFFSET UPSTREAM
50
40
10
5
F-Line
E-Line
D-Line
AB-Line
13+02.5
19+22.5
12+17.5
12+22.5
13+00
14+27
14+30
14+00
12+00
MATCH LINE B
25
20
15
Diversion Channel Line
9+60
9+85
20+00
NOTES
LEGEND
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
PLAN OF AS-BUILT GROUT CURTAIN
GEORGIA EARTH EMBANKMENT
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
84
REVISIONS

2


OFFSET UPSTREAM
OFFSET DOWNSTREAM
Grout curtain originates at station 42+37.5 (A-Line)
EAST END OF CONCRETE DAM 42+37.5
A-Line
B-Line
C-Line
43+00
LEGEND



U. S. ARMY ENGINEER DISTRICT, SAVANNAH
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SAVANNAH, GEORGIA

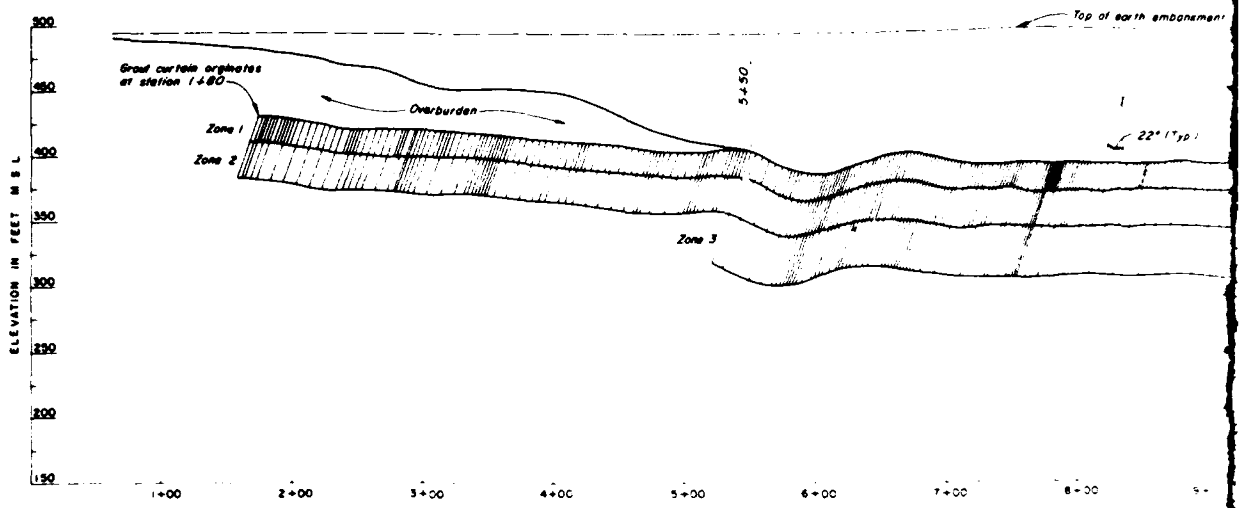
FOUNDATION REPORT
EMBANKMENTS

PLAN OF AS-BUILT GROUT CURTAIN
SOUTH CAROLINA EARTH EMBANKMENT

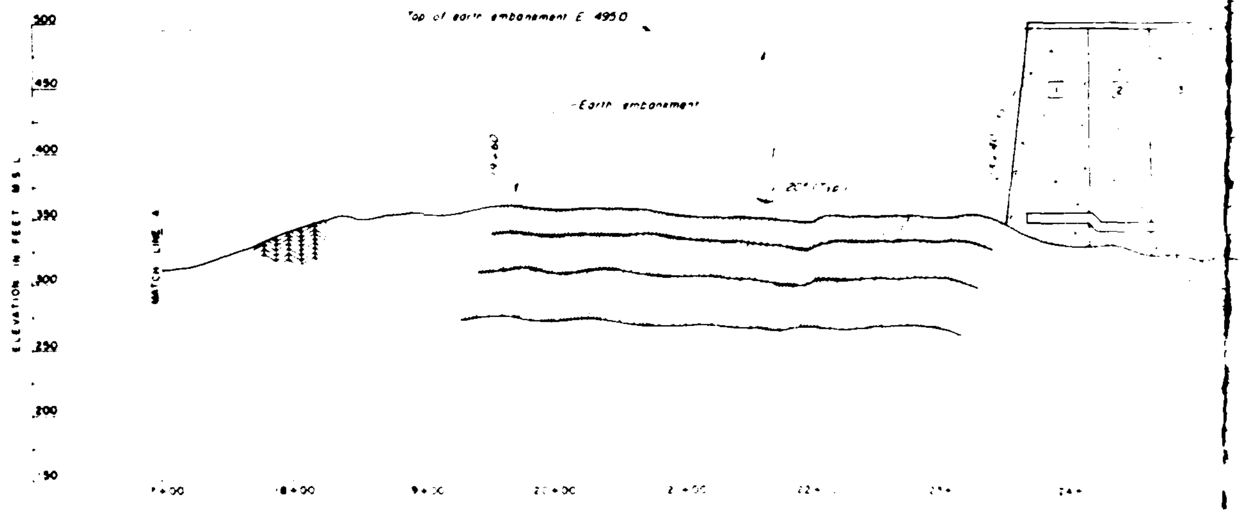
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 85

REVISIONS



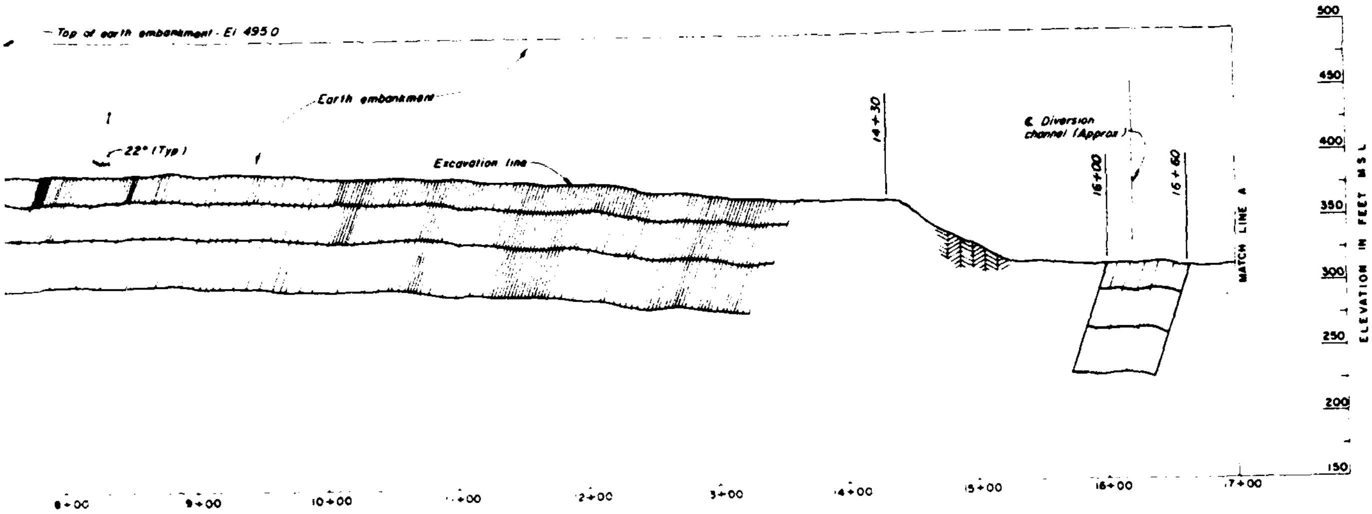
A-LINE CURTAIN GROUTING PROFILE



A-LINE CURTAIN GROUTING PROFILE

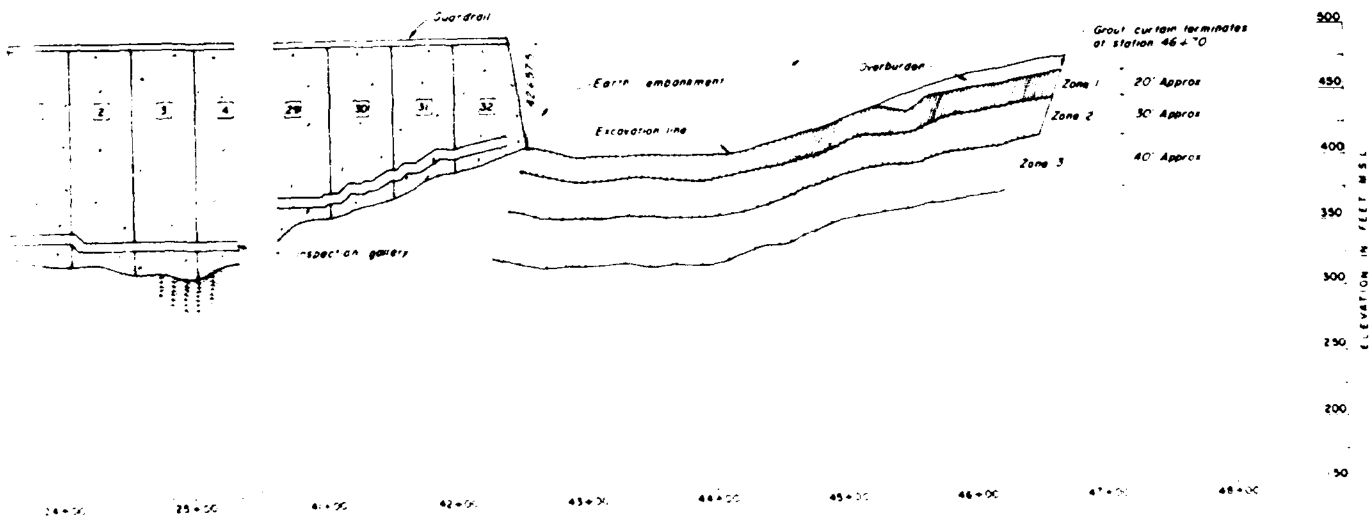
NOTES

1. For location of grout lines with respect to dam centerline in plan view, refer to Plates 84 and 85
2. For grout take data by zones, refer to Plates 93 thru 97
3. For details of drilling and grouting procedures used refer to Plates 110 & 111
4. For "As designed" contract grouting details refer to Plates 82 and 83
5. Note 20° westerly grout hole inclination. This attitude was most often used for intercepting the most frequently occurring set of fractures. (See geologic map Plate 5 this report for rose diagrams of fracture sets.
6. Note variation from original grouting scheme as determined by split spacing of "high take" holes and other field criteria
7. Water pressure testing data, communication between holes and surface leakage information are recorded on field grouting records and maintained in the files of SASEN G.



...TAIN GROUTING PROFILE – GEORGIA EARTH EMBANKMENT

SCALE ... 50



...NG PROFILE – GEORGIA EAST AND SOUTH CAROLINA EARTH EMBANKMENTS

U.S. ARMY ENGINEER DISTRICT, SAVANNAH
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SAVANNAH, GEORGIA

FOUNDATION REPORT
EMBANKMENTS

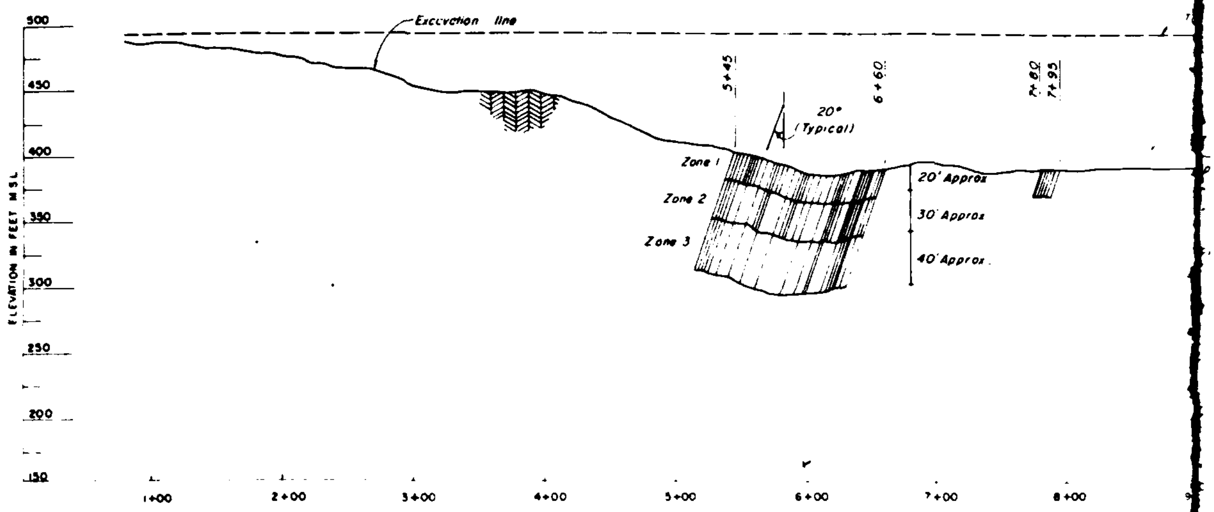
A-LINE CURTAIN GROUTING PROFILE-GA. & SOUTH CAROLINA EARTH EMBANKMENTS

RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

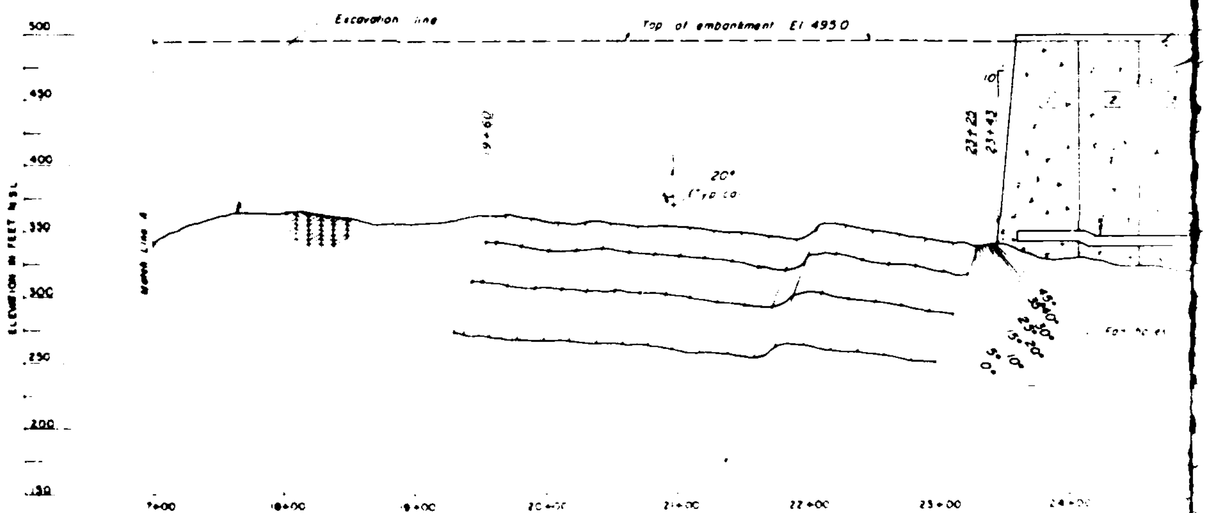
REVISIONS

PLATE 86

2



B-LINE CURTAIN GROUTING PROFILE



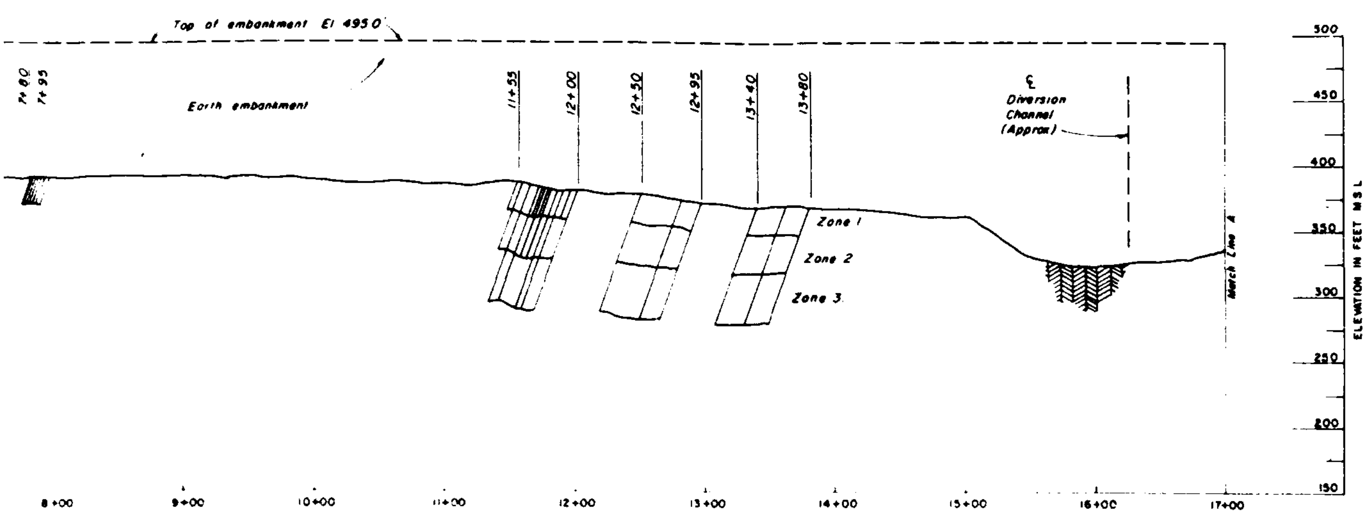
B-LINE CURTAIN GROUTING PROFILE - [illegible]EORGIA

NOTES

1) For location of grout line with respect to dam centerline in plan view, refer to Plates 84 and 85

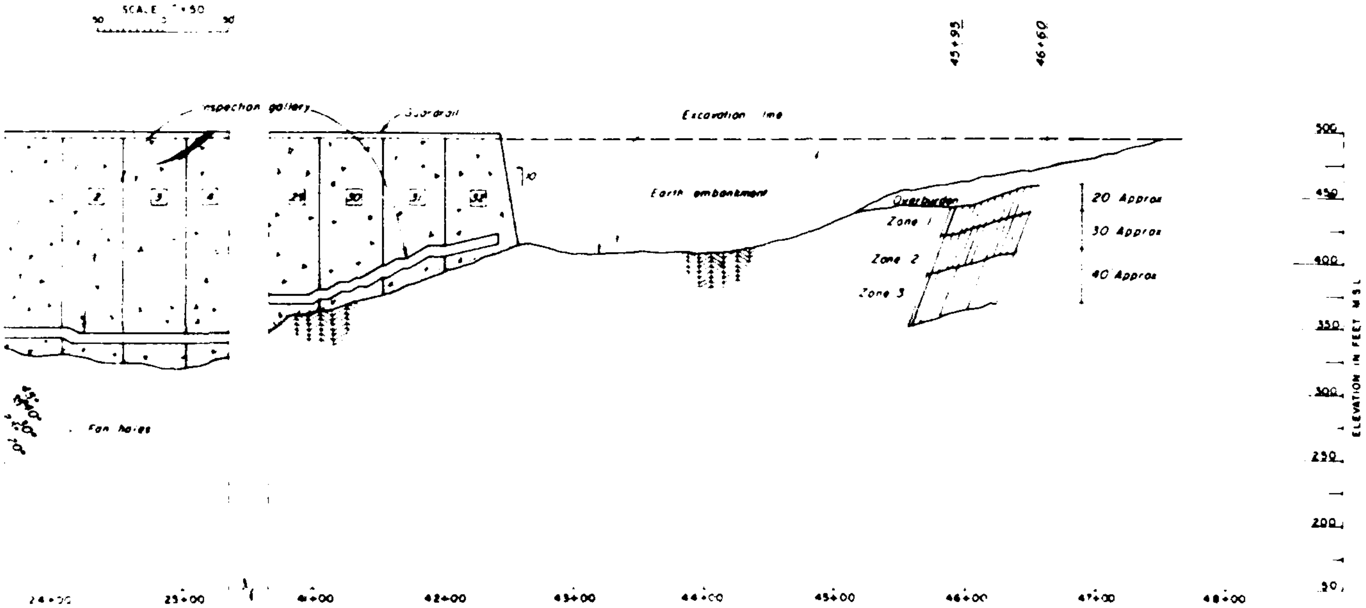
2) For grout-take data by zones, refer to Plates 98 thru 101

3) For details of drilling and grouting procedures used, refer to Plates 110 and 111



GROUTING PROFILE - GEORGIA EARTH EMBANKMENT

SCALE 1"=50'



ROFILE - GEORGIA EAST AND SOUTH CAROLINA EARTH EMBANKMENTS

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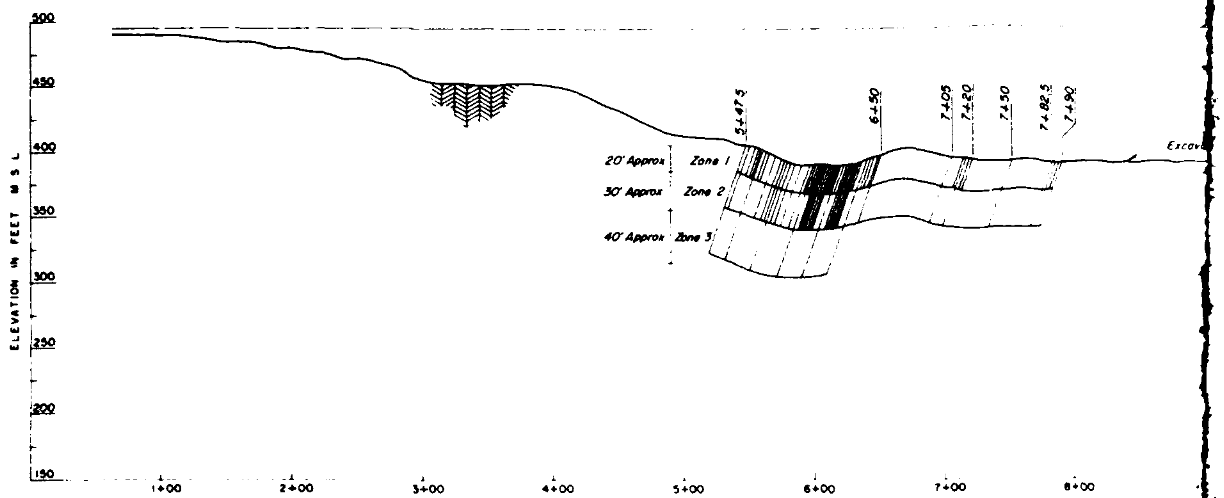
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
B-LINE CURTAIN GROUTING PROFILE-GA. & SOUTH CAROLINA EARTH EMBANKMENTS
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

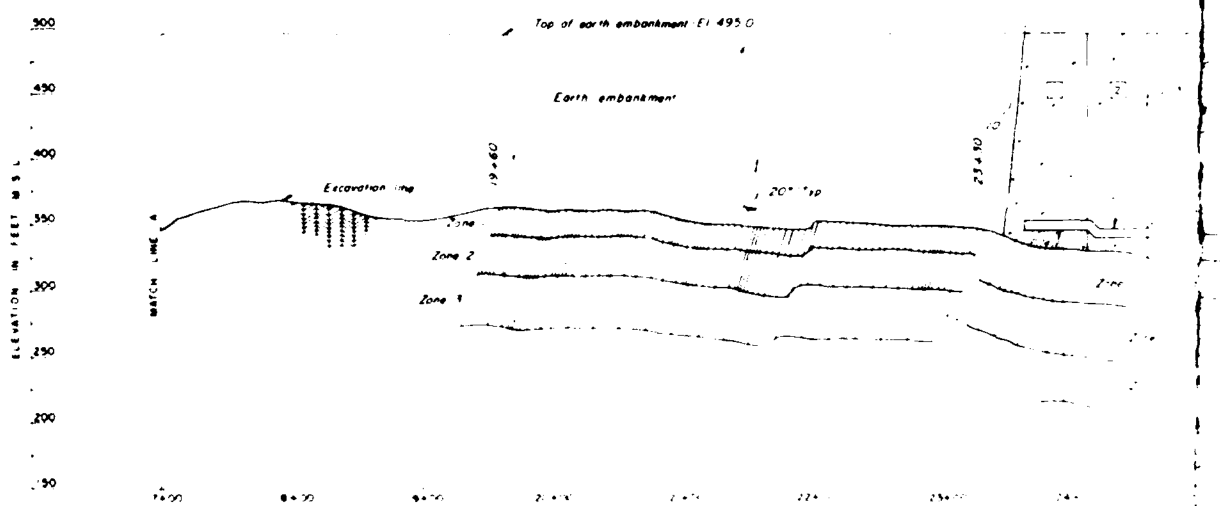
SCALE AS SHOWN

PLATE 87

REVISIONS



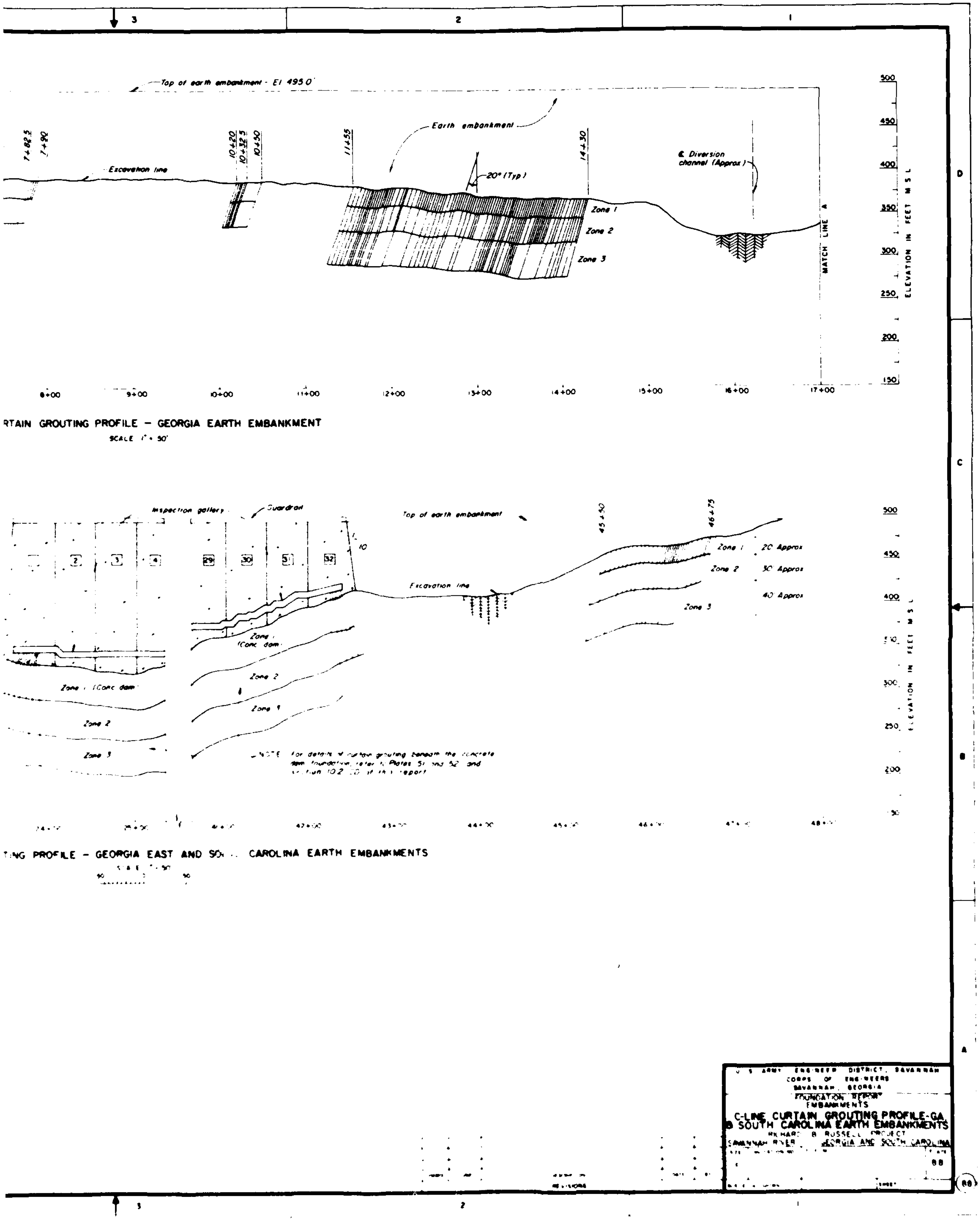
C-LINE CURTAIN GROUTING PROFI



C-LINE CURTAIN GROUTING PROFILE – GE

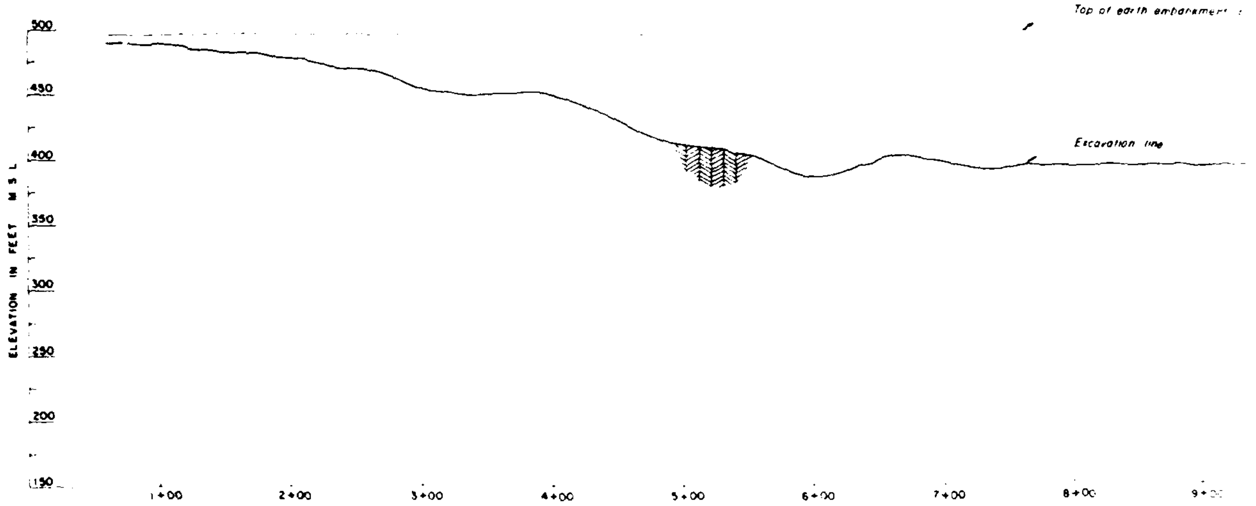
NOTES

1. For location of grout lines with respect to dam centerline in plan view, refer to Plates B4 and B5.
2. For grout take data by zones, refer to Plates C2 thru C5.
3. For details of drilling and grouting procedures used refer to Plates 10 B-11.

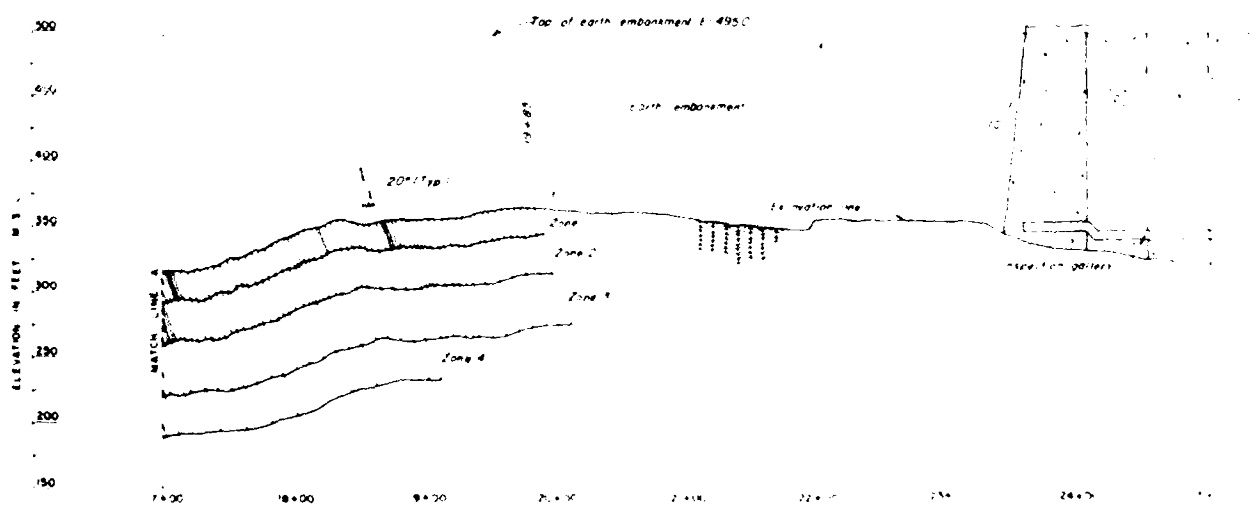


Top of earth embankment - El 495.0'
7+82.5
7+90
10+20
10+32.5
10+50
11+55
14+30
Earth embankment
Excavation line
20° (Typ)
℄ Diversion channel (Approx.)
Zone 1
Zone 2
Zone 3
MATCH LINE A
ELEVATION IN FEET M.S.L.
500
450
400
350
300
250
200
150
9+00
10+00
11+00
12+00
13+00
14+00
15+00
16+00
17+00
RTAIN GROUTING PROFILE – GEORGIA EARTH EMBANKMENT
Inspection gallery
Guardrail
Top of earth embankment
45+50
46+75
20 Approx
30 Approx
40 Approx
Excavation line
2
3
4
29
30
31
32
Zone 1 (Conc dam)
Zone 2
Zone 3
NOTE: For details of curtain grouting beneath the concrete dam foundation, refer to Plates 51 and 52 and section 10.2 of this report
42+00
43+00
44+00
45+00
46+00
48+00
ING PROFILE – GEORGIA EAST AND SO... CAROLINA EARTH EMBANKMENTS
U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
C-LINE CURTAIN GROUTING PROFILE-GA. & SOUTH CAROLINA EARTH EMBANKMENTS
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
88

2



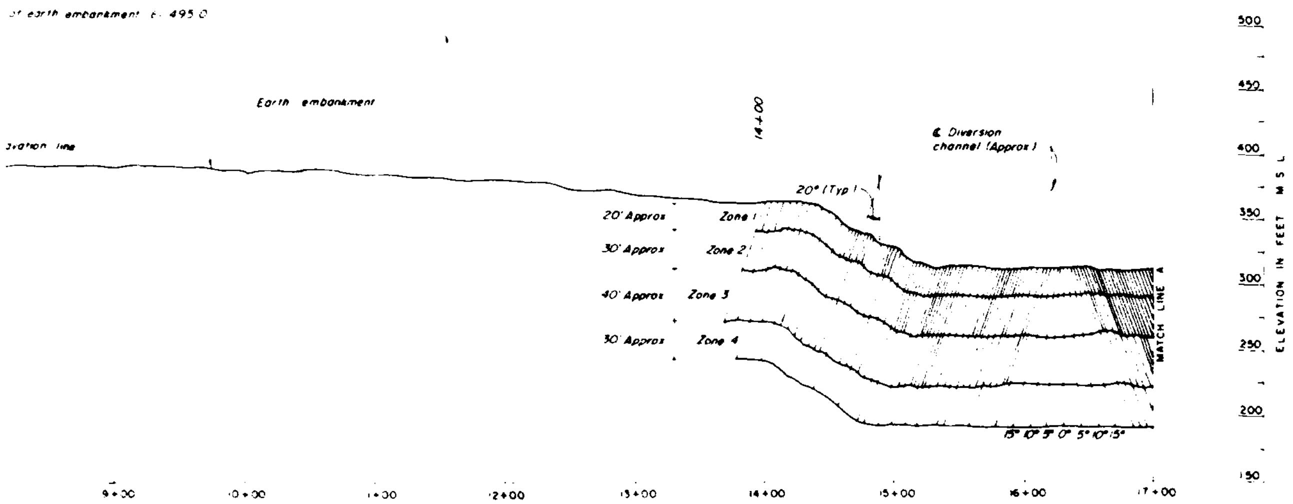
CURTAIN GROUTING PROFILE OF DIVERSION-



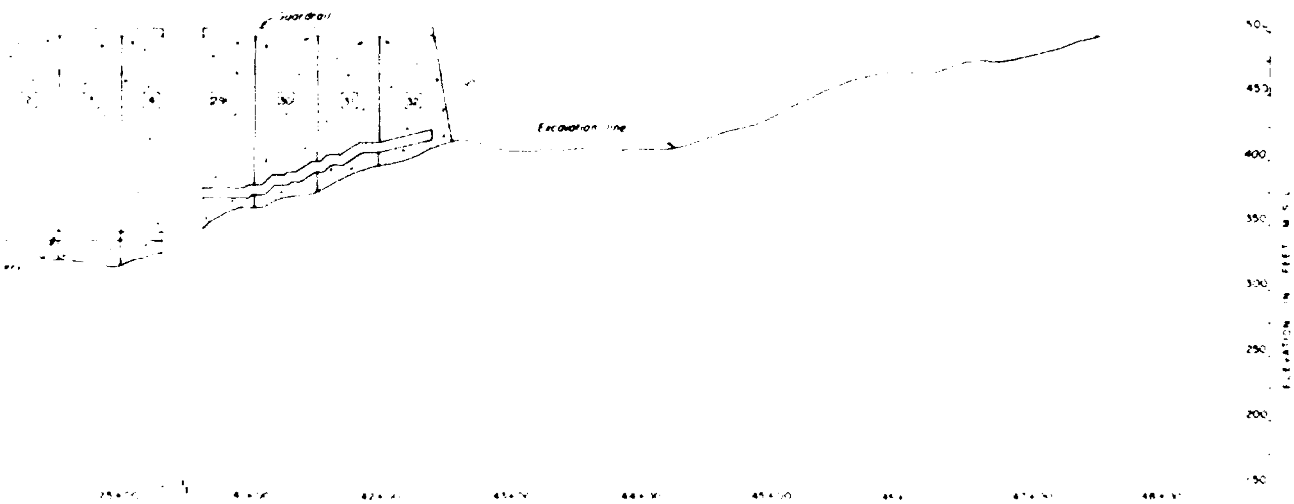
CURTAIN GROUTING PROFILE OF DIVERSION-CHANNEL LINE

NOTES

1. For location of grout lines with respect to dam centerline in plan view, refer to Plates 84 and 85
2. For grout take data by zones, refer to Plate 106
3. For details of drilling and grouting procedures used, refer to Plates 110 & 111



FILE OF DIVERSION-CHANNEL LINE - GEORGIA EMBANKMENT
SCALE 1" = 50'



IN-CHANNEL LINE - GEORGIA AND SOUTH CAROLINA EARTH EMBANKMENTS

U.S. ARMY ENGINEER DISTRICT, SAVANNAH
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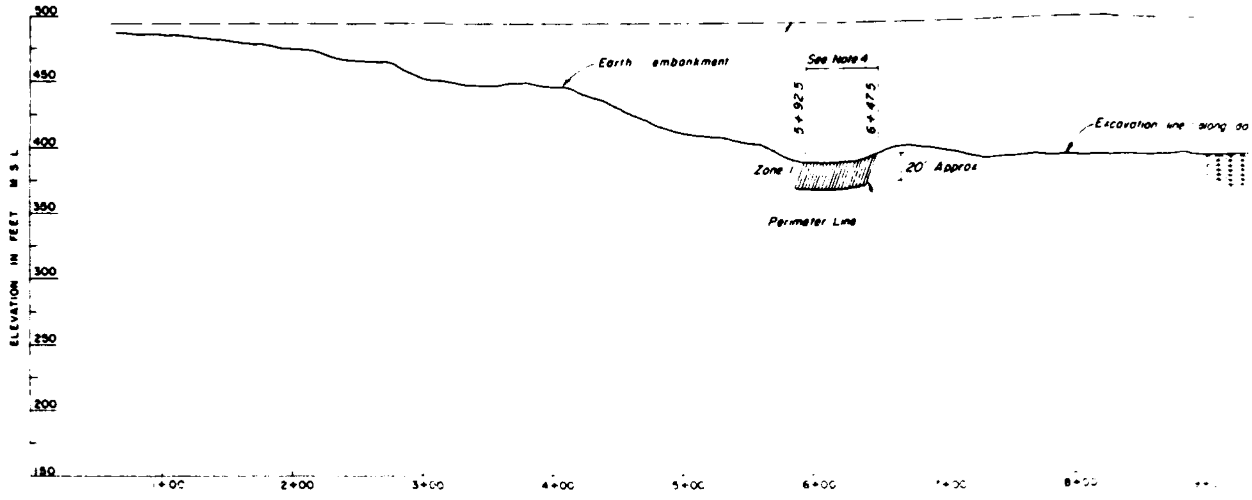
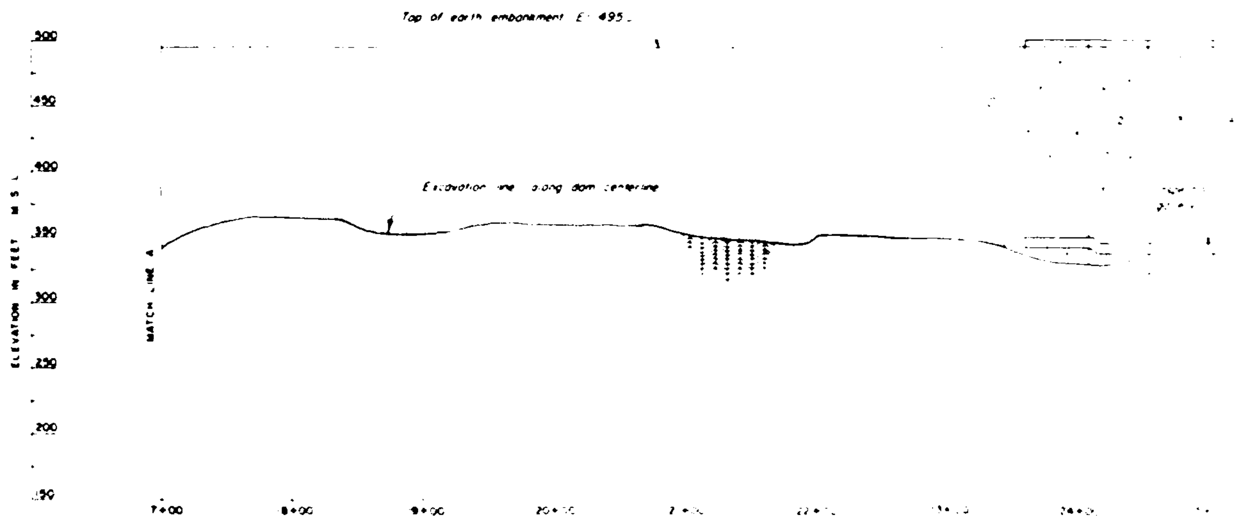
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
CURTAIN GROUTING PROFILE OF DIVERSION-CHANNEL LINE-GEORGIA EMBANKMENT
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

89

REVISIONS



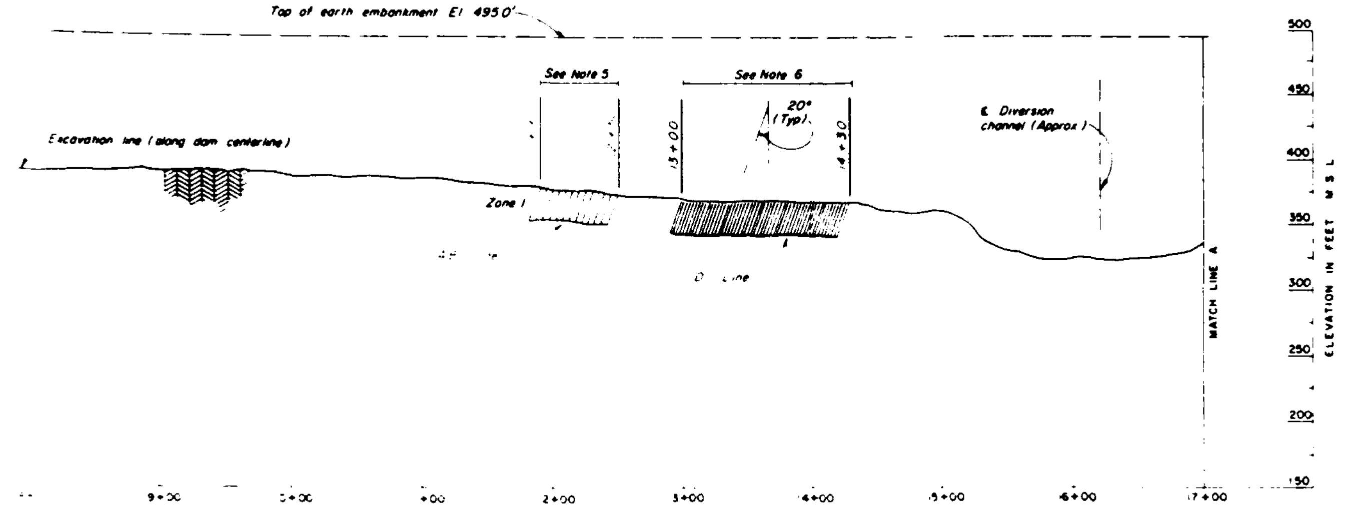
CURTAIN GROUTING PROFILE OF PERIMETER LINE, AB-L



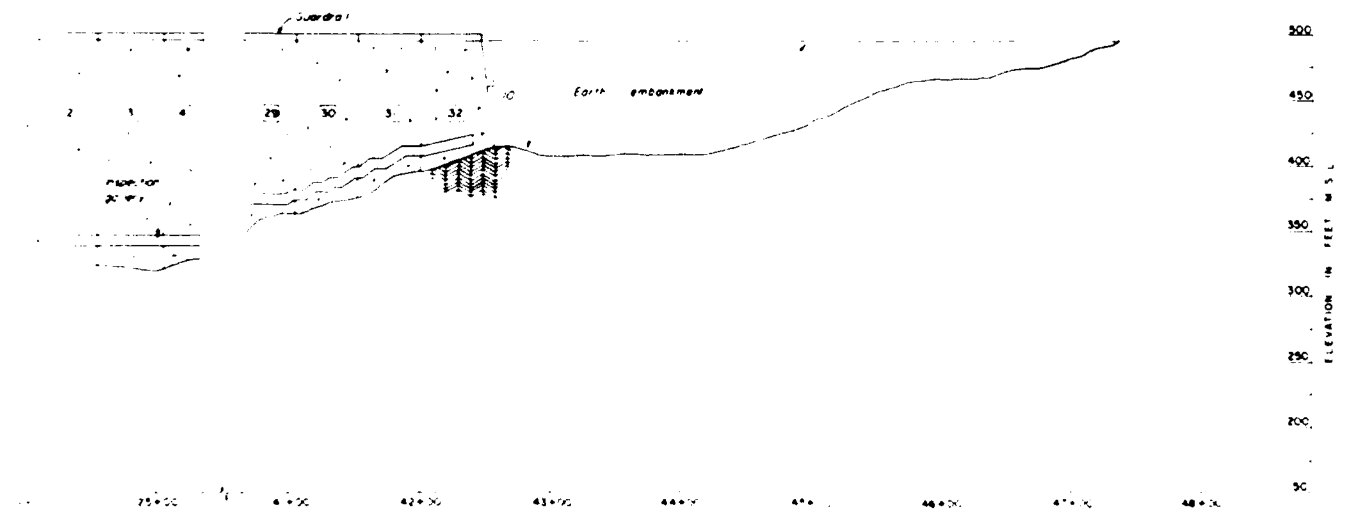
CURTAIN GROUTING PROFILE OF PERIMETER LINE, AB LINE B

NOTES

1. For location of individual grout lines with respect to dam centerline in plan view, refer to Plates 84 and 85
2. For grout take data by zones, refer to Plate 107
3. For details of drilling and grouting procedures used, refer to Plates 110 and 111
4. Perimeter Line is located 40-50 feet upstream from dam centerline
5. AB - Line is located 75 feet upstream from dam centerline
6. D - Line is located 50 feet upstream from dam centerline



…RMETER LINE, AB-LINE & D-LINE - GEORGIA EARTH EMBANKMENT



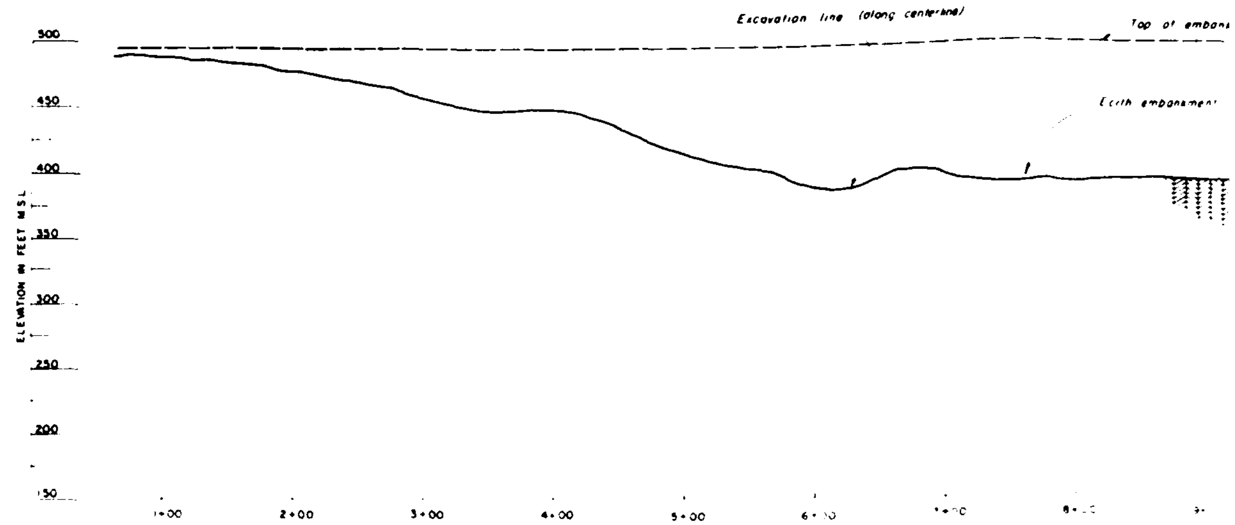
…NE, AB-LINE & D-LINE - GEORGIA AND SOUTH CAROLINA EARTH EMBANKMENTS

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

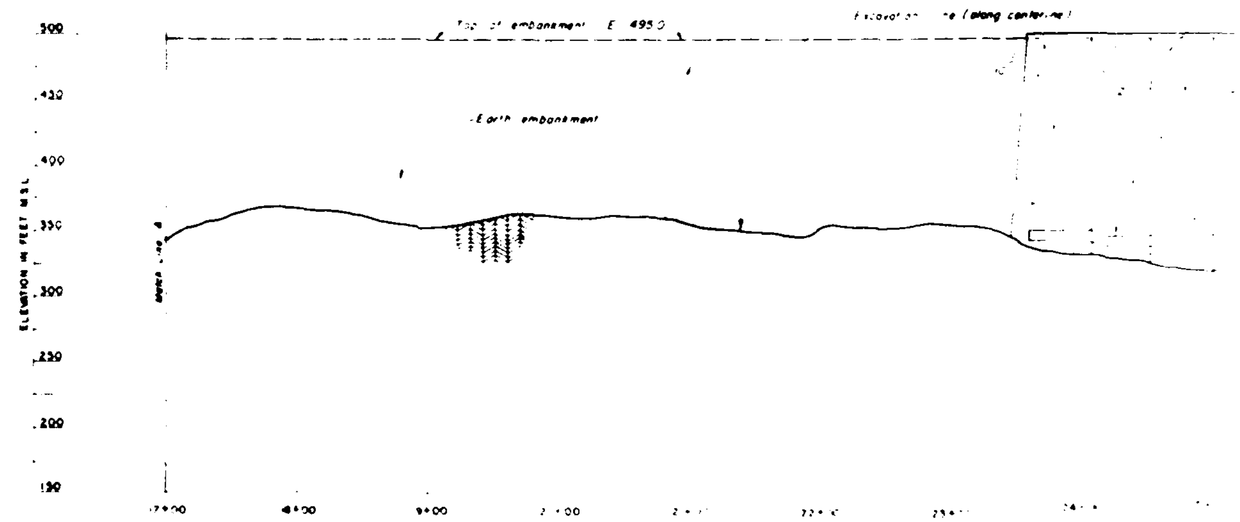
FOUNDATION REPORT-EMBANKMENTS
CURTAIN GROUTING PROFILE OF PERIMETER LINE, AB-LINE & D-LINE - GEORGIA AND SOUTH CAROLINA EARTH EMBANKMENTS
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 90

2



E-LINE CURTAIN GROUTING PROFILE -



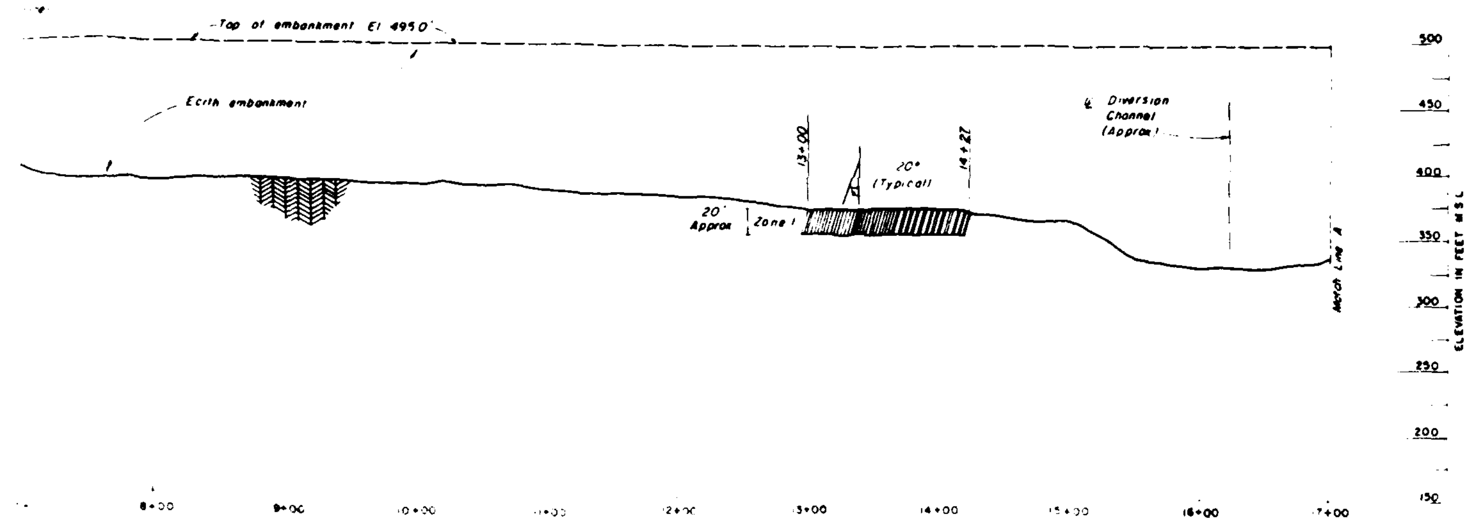
E-LINE CURTAIN GROUTING PROFILE - GEORGIA AN

NOTES:

1) For location of grout line with respect to dam centerline in plan view, refer to Plates 84 and 85

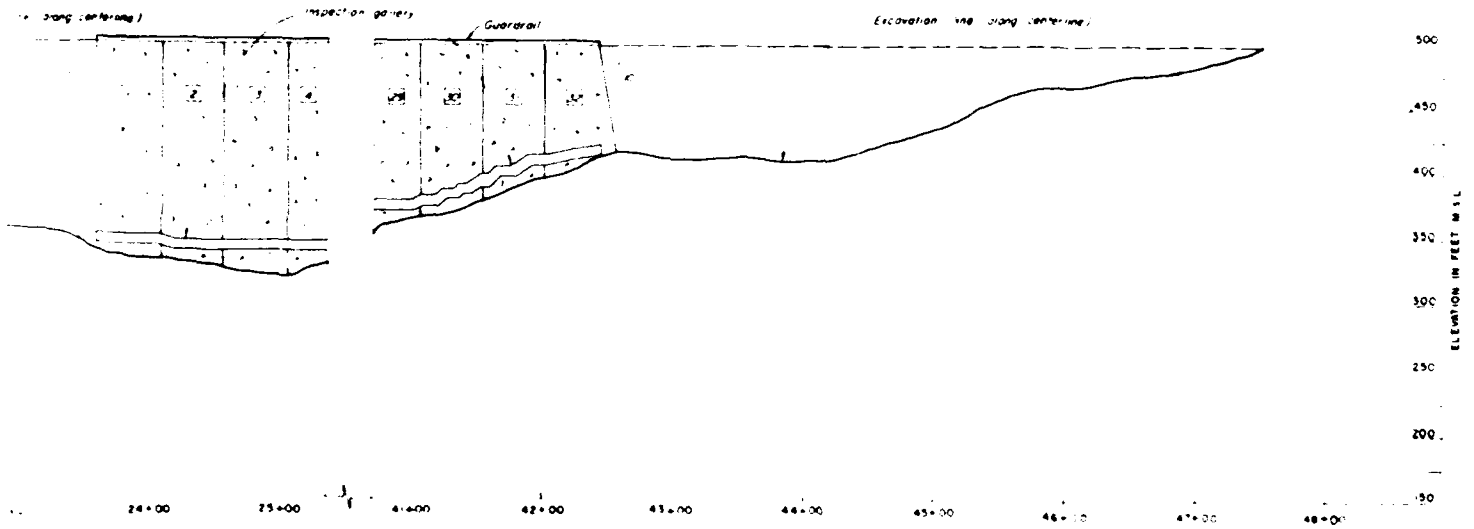
2) For grout take data by zones, refer to Plate 107

3) For details of drilling and grouting procedures used, refer to Plates 110 and 111



CURTAIN GROUTING PROFILE - GEORGIA EARTH EMBANKMENT

SCALE 1"=50'



GROUTING PROFILE - GEORGIA AND SOUTH CAROLINA EARTH EMBANKMENTS

SCALE 1"=50'

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
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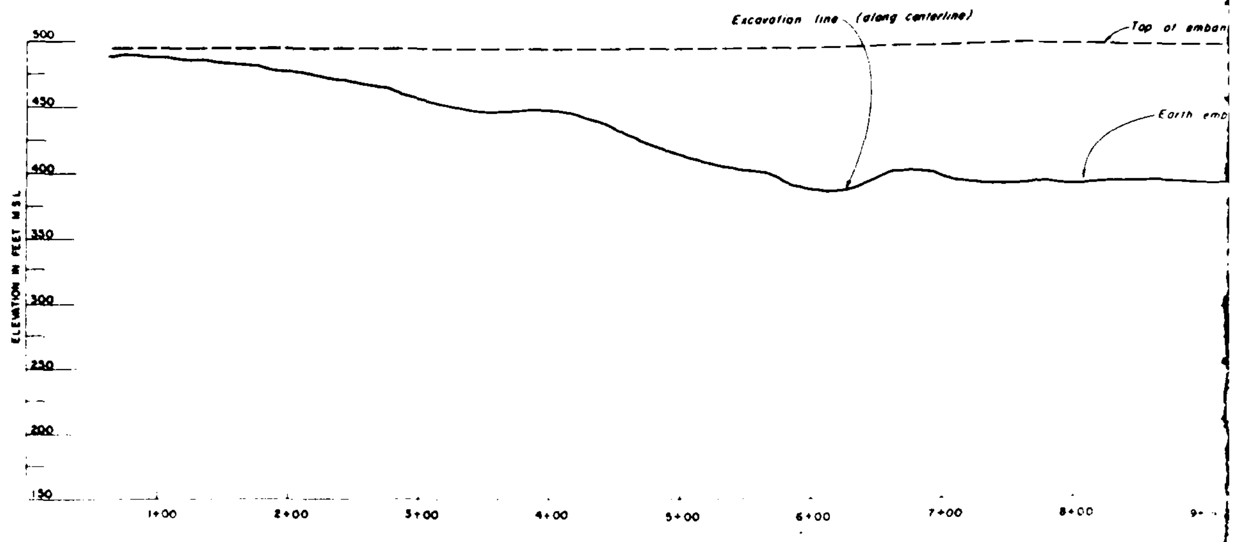
SAVANNAH, GEORGIA

FOUNDATION REPORT
EMBANKMENTS
E-LINE CURTAIN GROUTING PROFILE-GEORGIA
& SOUTH CAROLINA EARTH EMBANKMENTS
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

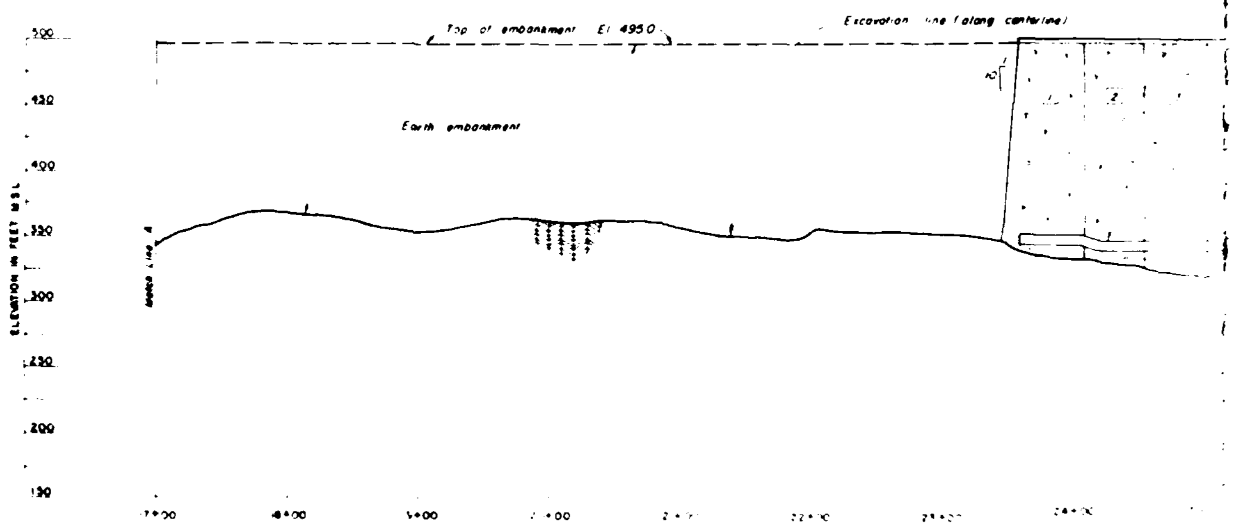
PLATE 91

REVISIONS

91



F-LINE CURTAIN GROUTING PROFILE -



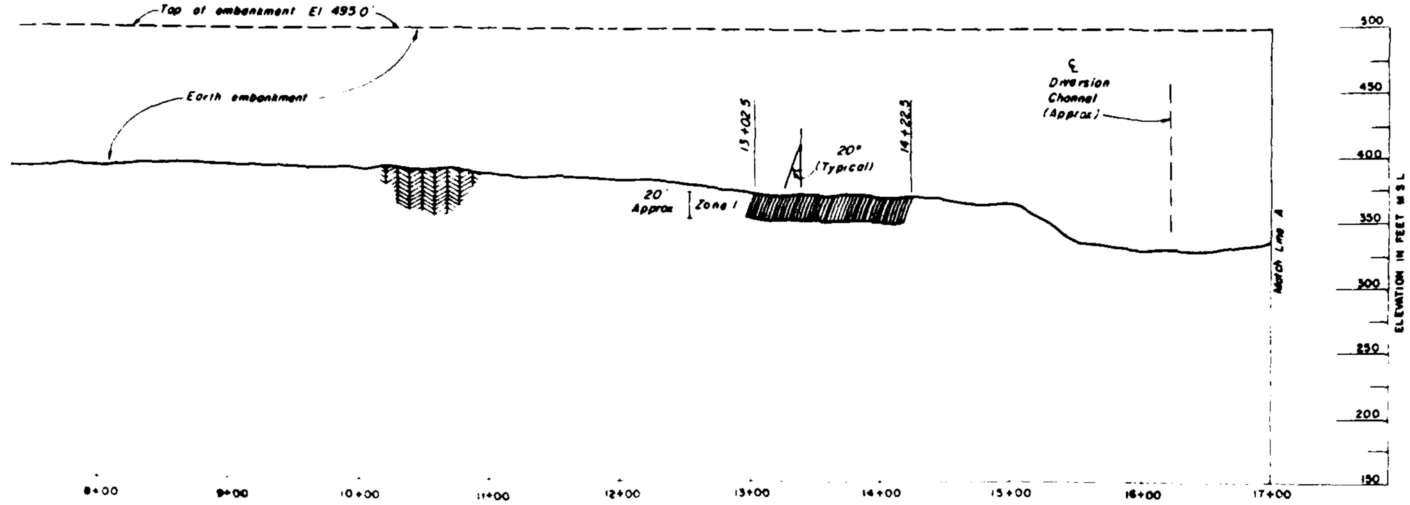
F-LINE CURTAIN GROUTING PROFILE - GEORGIA A

NOTES:

1) For location of grout line with respect to dam centerline in plan view, refer to Plates 84 and 85

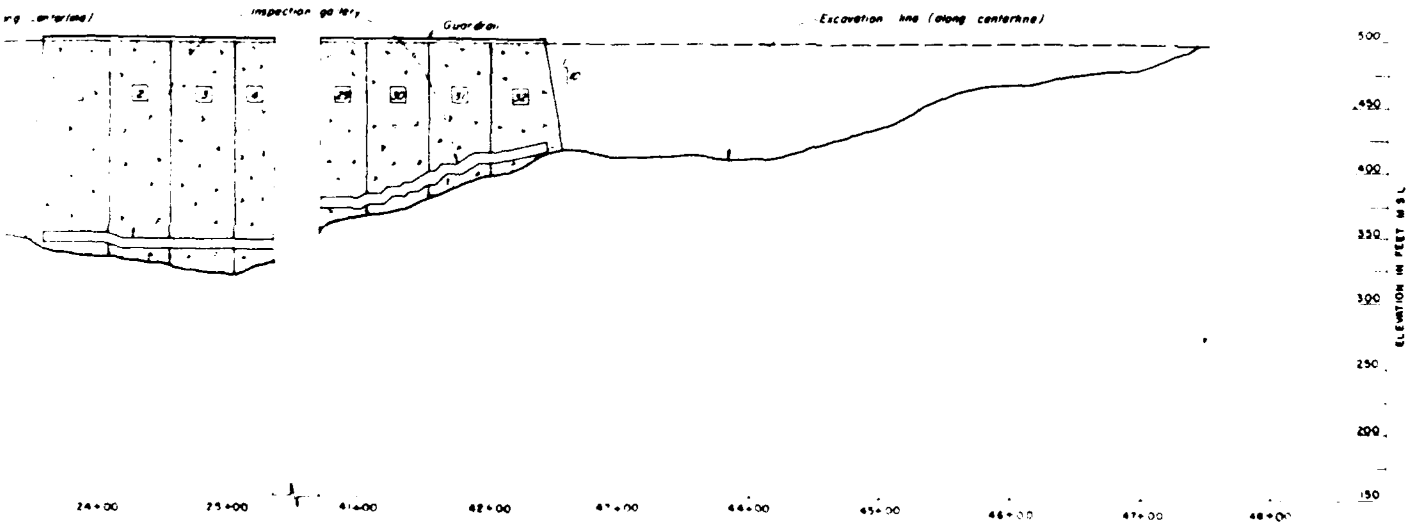
2) For grout take data by zones, refer to Plate 107

3) For details of drilling and grouting procedures used, refer to Plates 110 and 111



TAIN GROUTING PROFILE - GEORGIA EARTH EMBANKMENT

SCALE 1" = 50'



TING PROFILE - GEORGIA AND SOUTH CAROLINA EARTH EMBANKMENTS

SCALE 1" = 50'

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
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SAVANNAH, GEORGIA

FOUNDATION REPORT
EMBANKMENTS

F-LINE CURTAIN GROUTING PROFILE - GEORGIA & SOUTH CAROLINA EARTH EMBANKMENTS

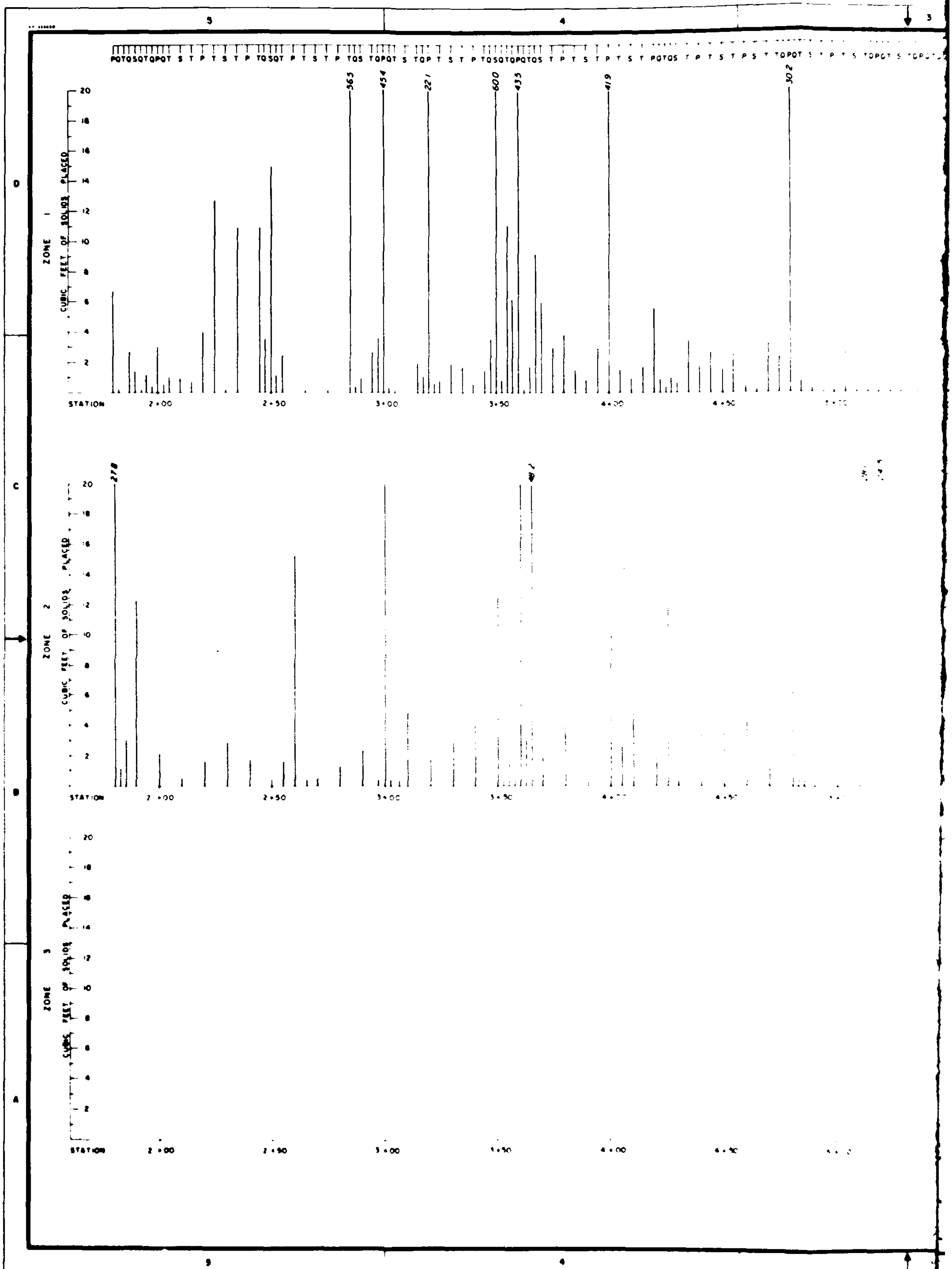
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 92

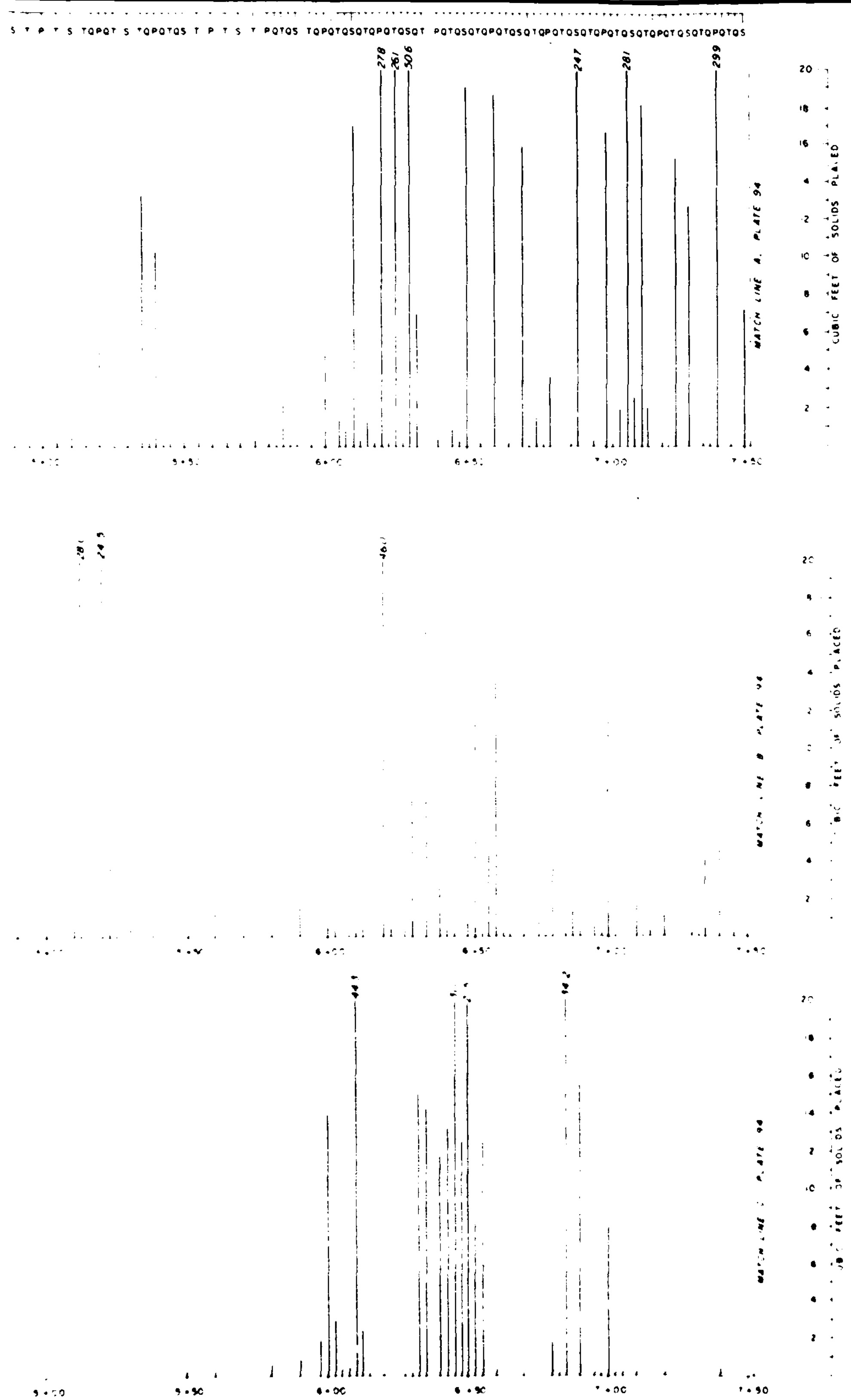
SCALE: AS SHOWN

REVISIONS

92



ZONE 1
CUBIC FEET OF SOLIDS PLACED
20
18
16
14
12
10
8
6
4
2
563
454
221
600
435
419
302
STATION
2+00
2+50
3+00
3+50
4+00
4+50
ZONE 2
CUBIC FEET OF SOLIDS PLACED
278
STATION
2+00
2+50
3+00
3+50
ZONE 3
CUBIC FEET OF SOLIDS PLACED
STATION
2+00
2+50
3+00
3+50
4+00



NOTES

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85
2. All grout holes on this plate are inclined in a 20° westerly direction
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-66 for a limited time

ZONE [illegible]
ZONE 2+2[illegible]
ZONE 3+50[illegible]

P = Primary grout hole
S = Secondary grout hole
T = Tertiary grout hole
Q = Quaternary grout hole

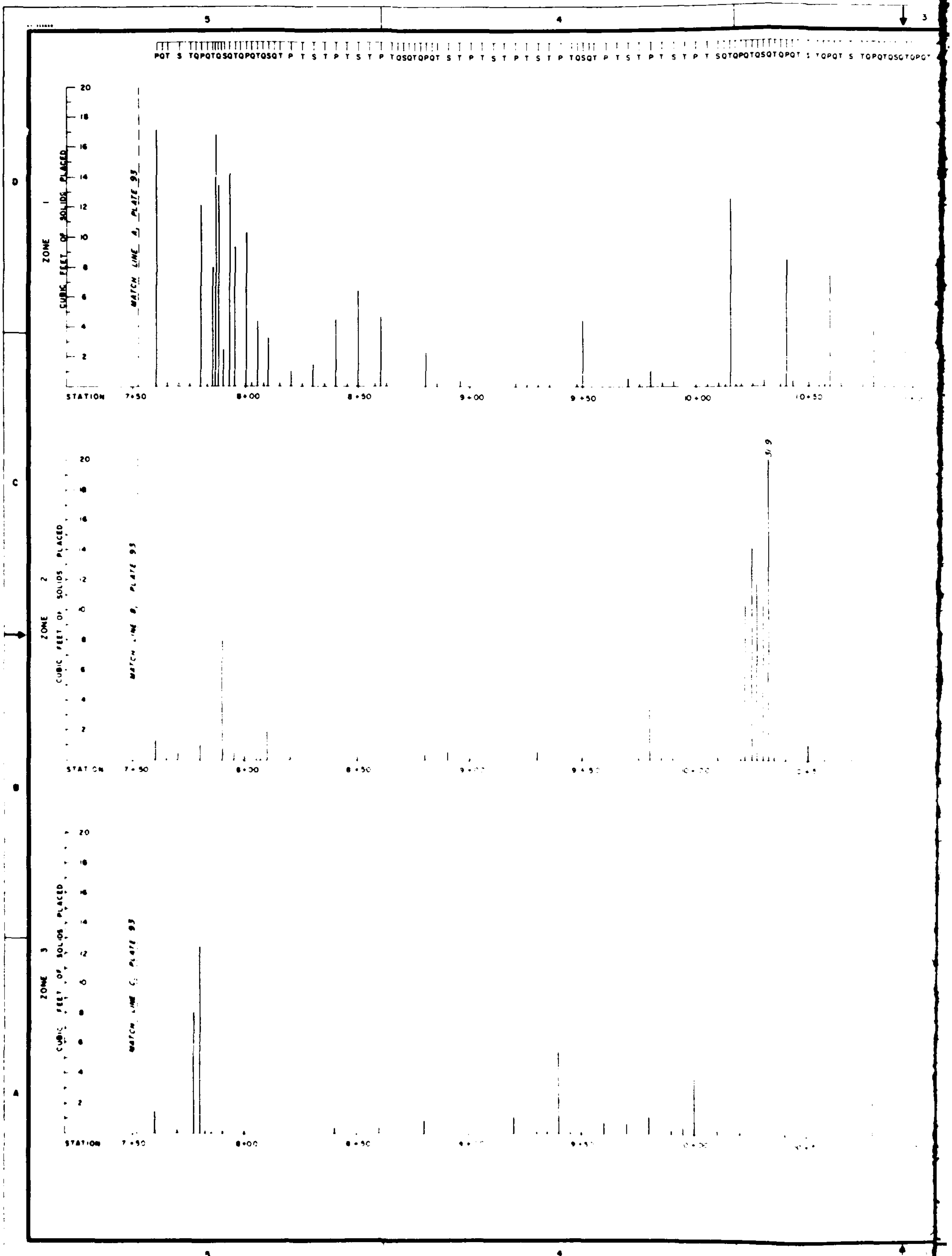
REVISIONS

U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

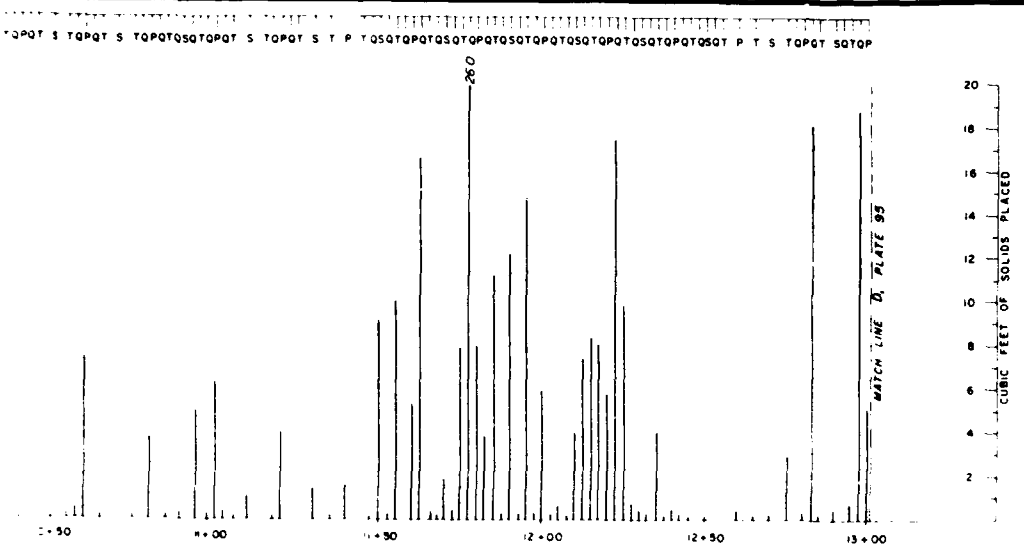
FOUNDATION REPORT - EMBANKMENTS
GROUT TAKES FOR GROUT LINE "A"
GA & SC EARTH EMBANKMENTS
(SHEET 1 OF 5)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

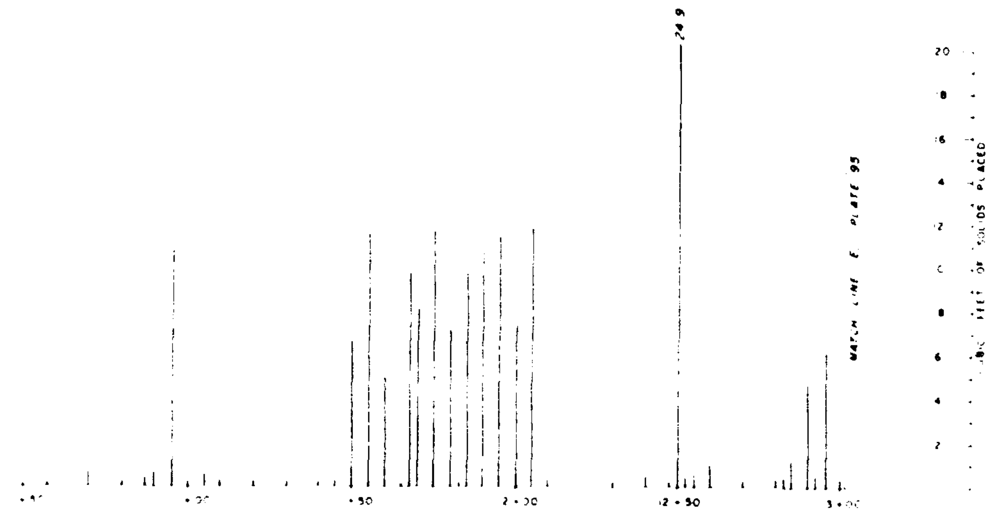
PLATE 93

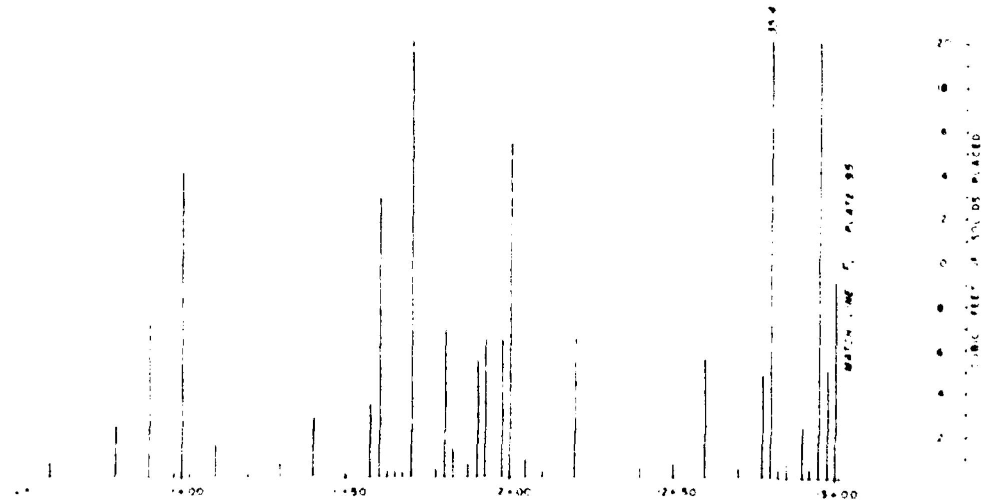
2



ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCH LINE A, PLATE 95
ZONE 2
CUBIC FEET OF SOLIDS PLACED
MATCH LINE B, PLATE 95
ZONE 3
CUBIC FEET OF SOLIDS PLACED
MATCH LINE C, PLATE 95
STATION
7+50
8+00
8+50
9+00
9+50
10+00
10+50
20
18
16
14
12
10
8
6
4
2







NOTES

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85

2. All grout holes on this plate are inclined in a 20° westerly direction

3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1 - 0-20
ZONE 2 - 20-50
ZONE 3 - 50-90

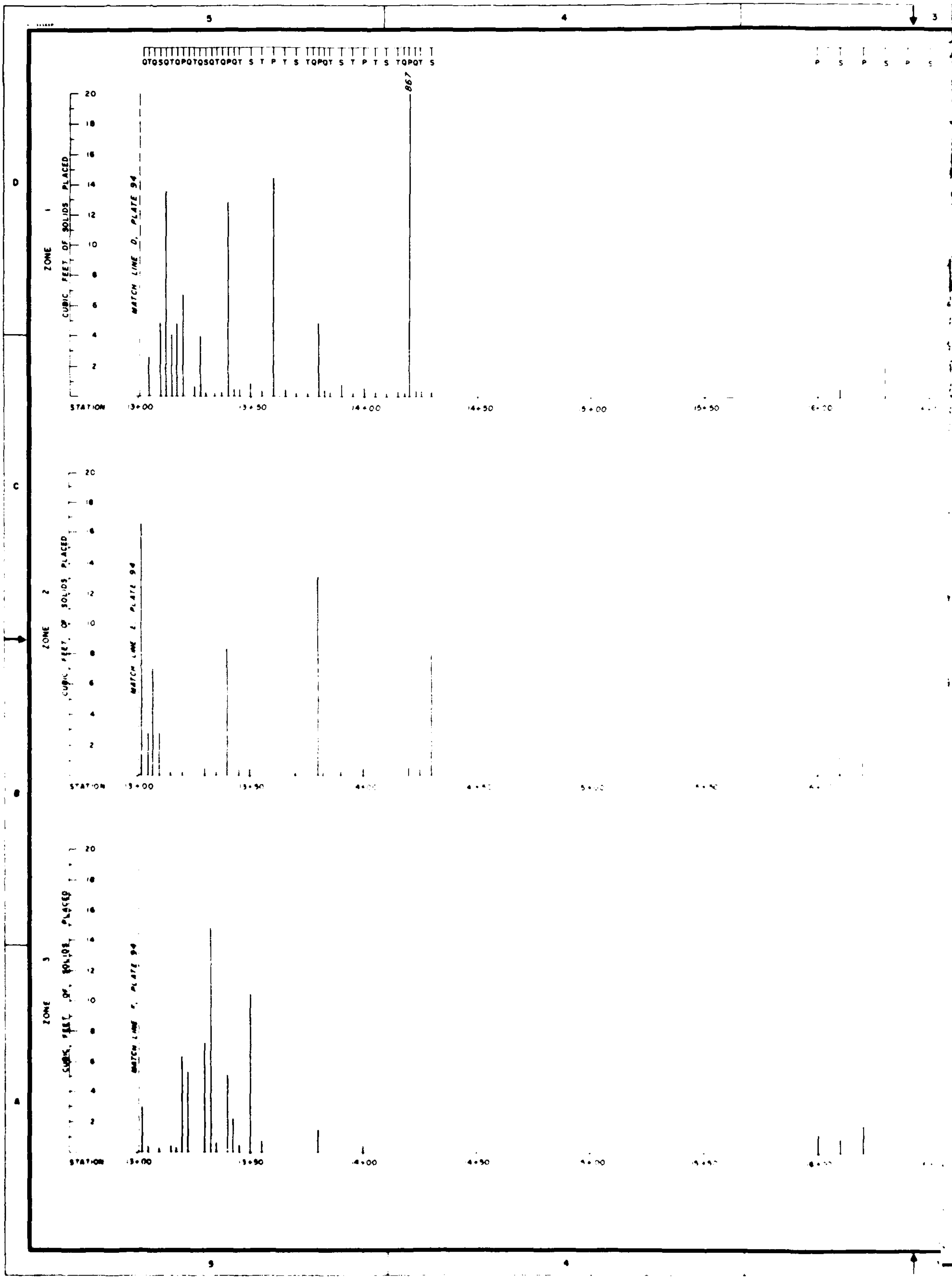
P - Primary grout hole
S - Secondary grout hole
T - Tertiary grout hole
Q - Quaternary grout hole

U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

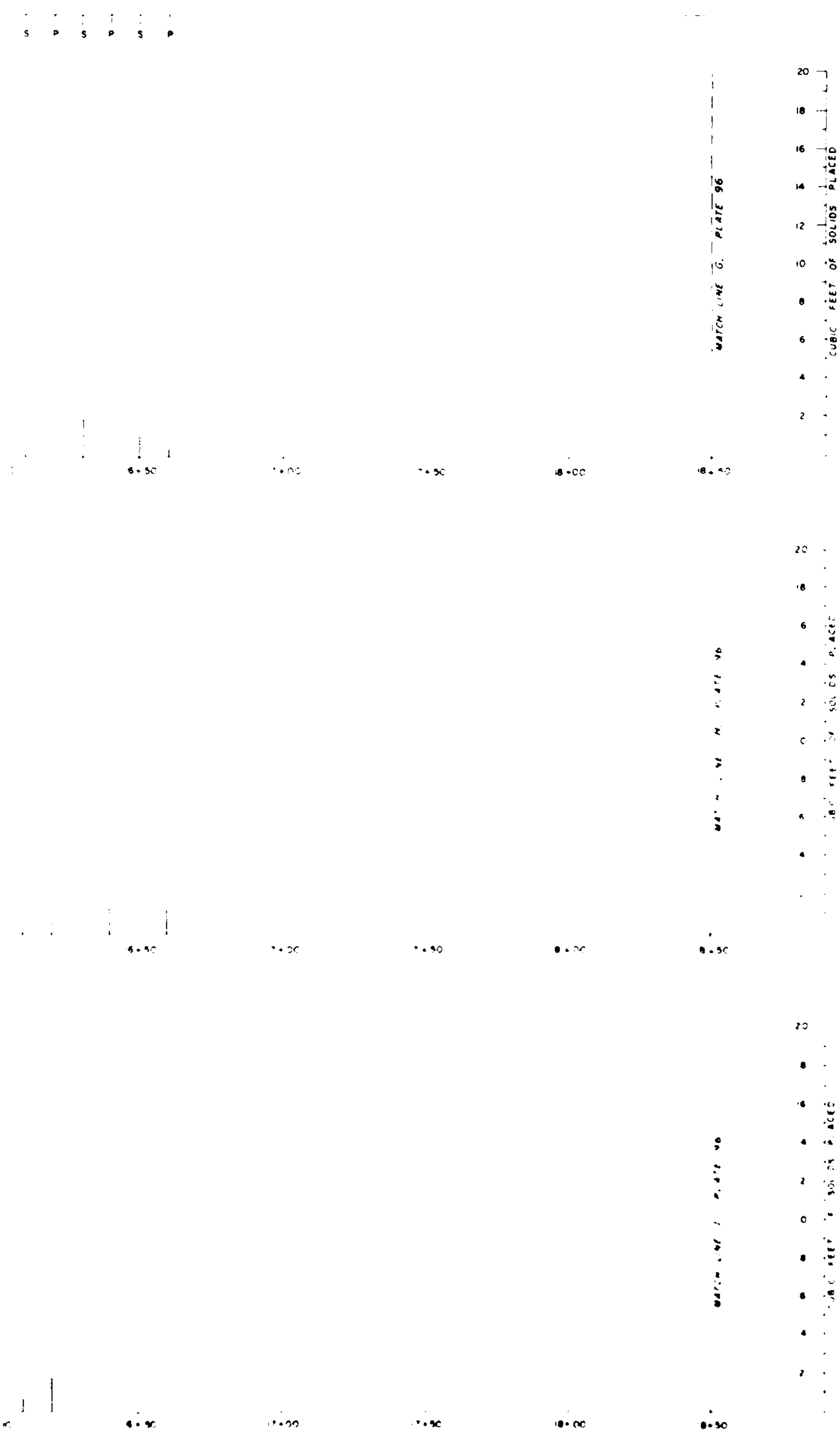
FOUNDATION REPORT EMBANKMENTS
GROUT TAKES FOR GROUT LINE "A"
GA & SC EARTH EMBANKMENTS
(SHEET 2 OF 5)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER, GEORGIA AND SOUTH CAROLINA

PLATE 94

2



ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCH LINE D, PLATE 94
867
ZONE 2
CUBIC FEET OF SOLIDS PLACED
MATCH LINE E, PLATE 94
ZONE 3
CUBIC FEET OF SOLIDS PLACED
MATCH LINE F, PLATE 94
STATION
13+00
13+50
14+00
14+50
15+00
15+50
16+00



NOTES

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85
2. All grout holes on this plate are inclined in a 20° westerly direction
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

P - Primary grout hole
S - Secondary grout hole
T - Tertiary grout hole
Q - Quaternary grout hole

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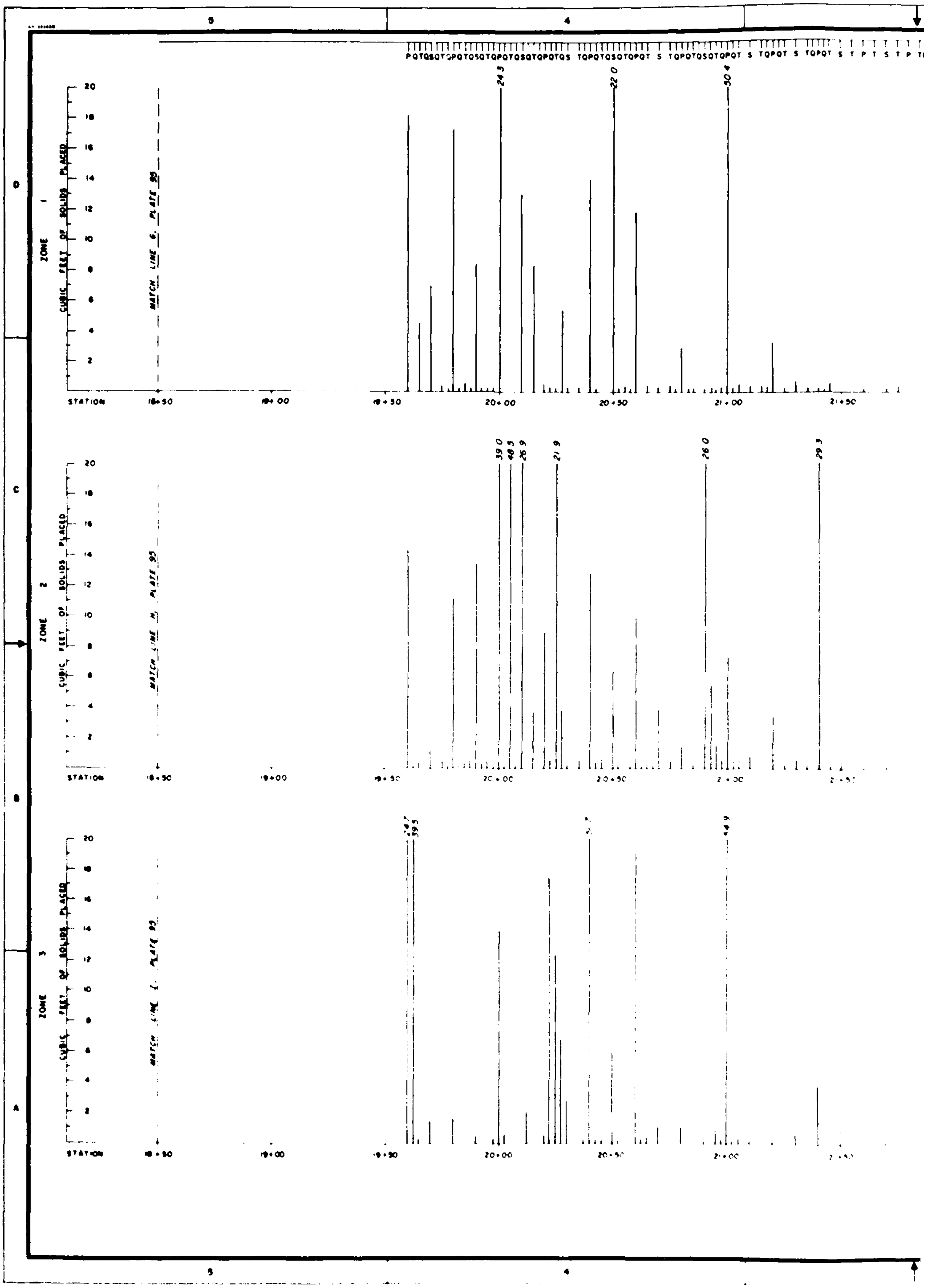
SAVANNAH, GEORGIA

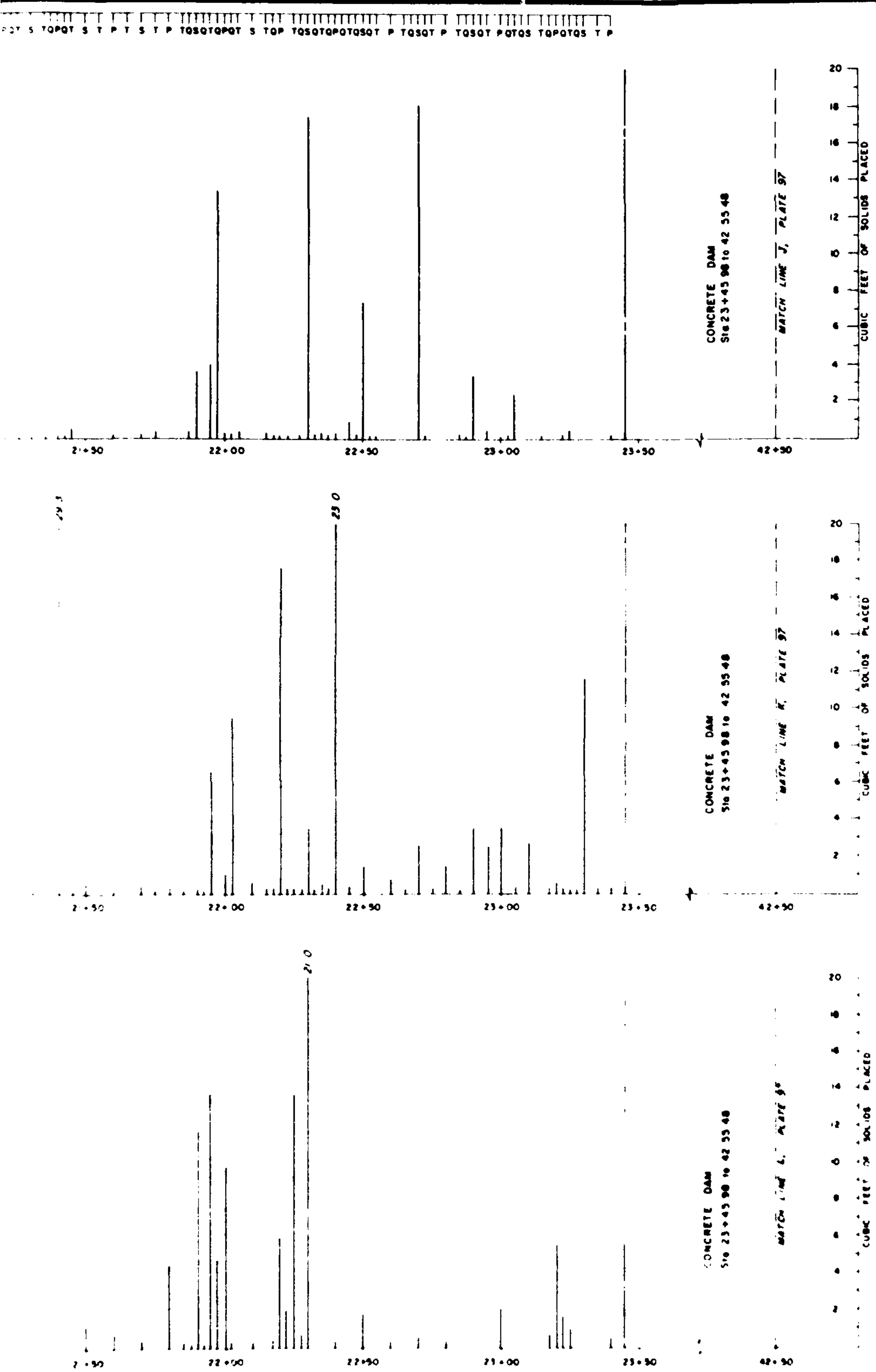
FOUNDATION REPORT - EMBANKMENTS
GROUT TAKES FOR GROUT LINE "A"
GA & SC EARTH EMBANKMENTS
(SHEET 3 OF 5)
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 95

SCALE AS SHOWN

2


ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCH LINE G, PLATE 95
ZONE 2
MATCH LINE H, PLATE 95
ZONE 3
MATCH LINE I, PLATE 95
STATION
18+50
19+00
19+50
20+00
20+50
21+00
21+50
24.3
22.0
50.4
39.0
48.5
26.9
21.9
26.0
29.3



NOTES:

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85
2. All grout holes on this plate are inclined in a 20° westerly direction
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1 - 0 - 20
ZONE 2 - 20 - 50
ZONE 3 - 50 - 90

P - Primary grout hole
S - Secondary grout hole
T - Tertiary grout hole
Q - Quaternary grout hole

U.S. ARMY ENGINEER DISTRICT, SAVANNAH
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SAVANNAH, GEORGIA

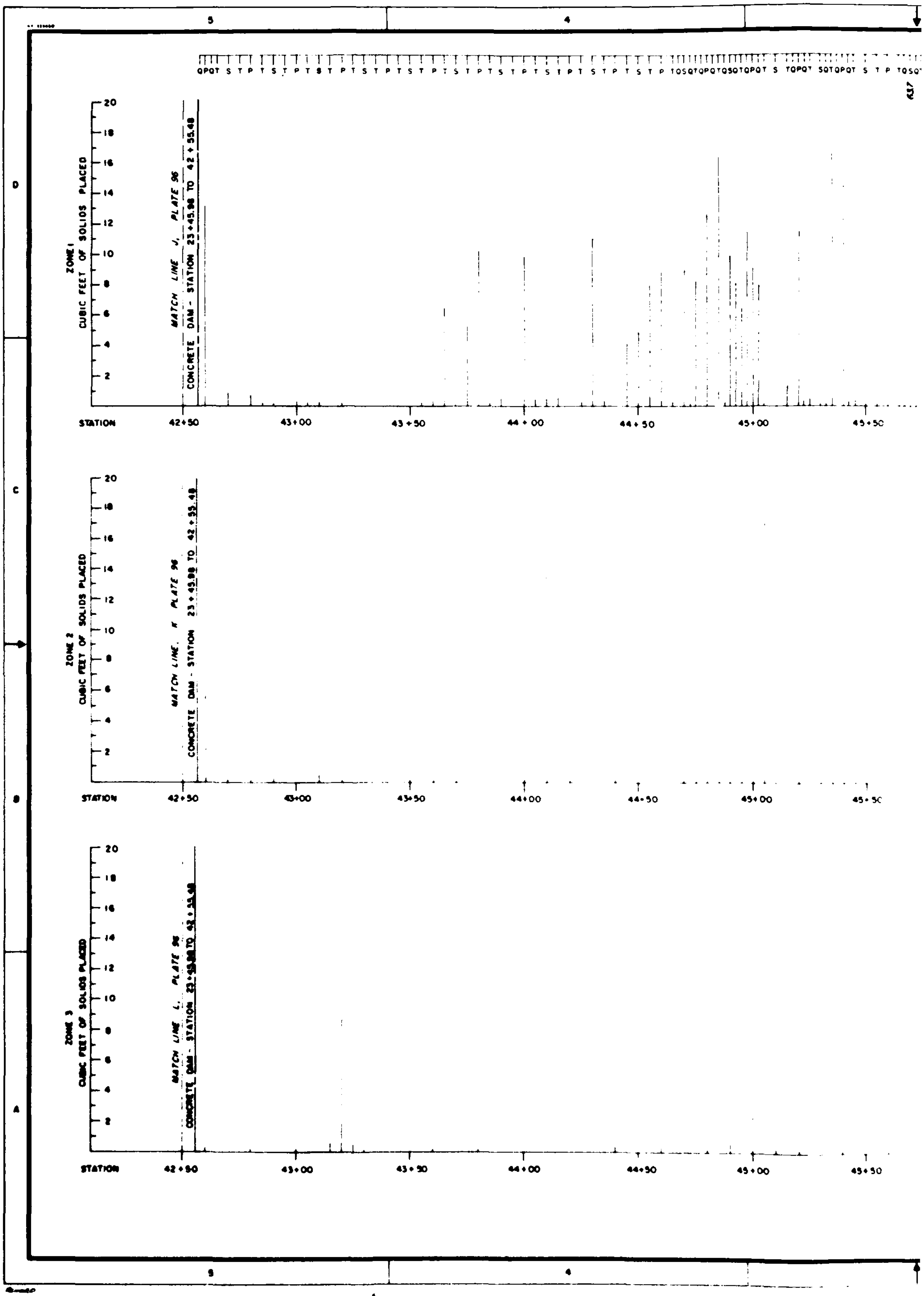
FOUNDATION REPORT - EMBANKMENTS
GROUT TAKES FOR GROUT LINE "A"
GA & SC EARTH EMBANKMENTS
(SHEET 4 OF 5)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

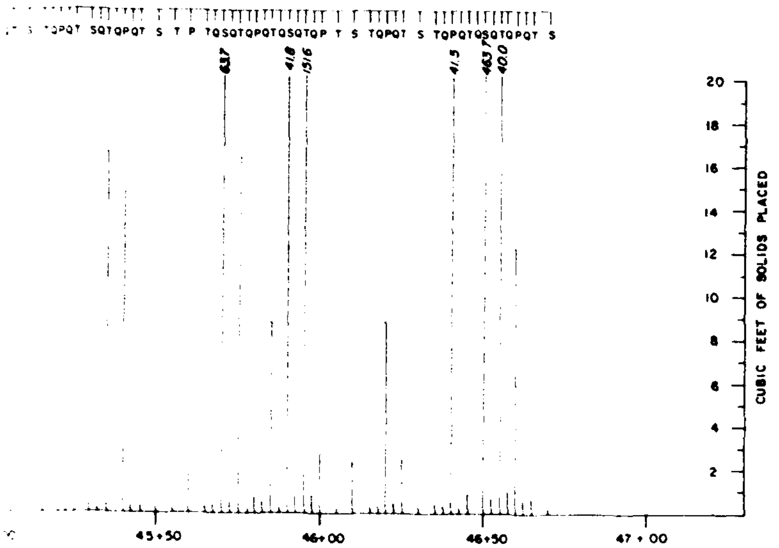
PLATE 96

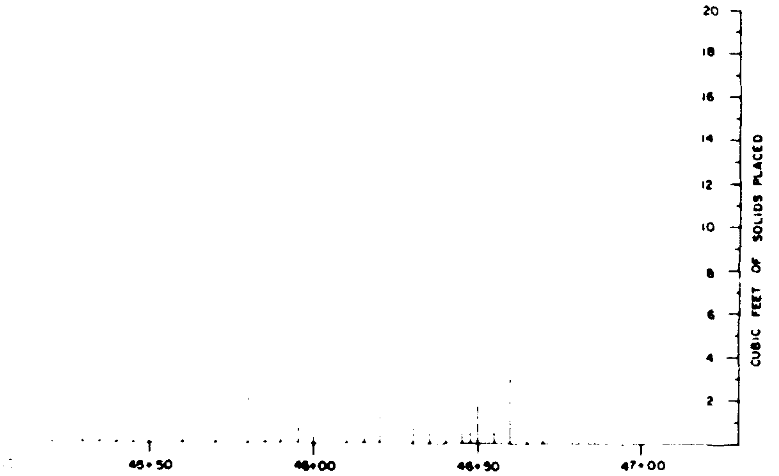
SCALE: AS SHOWN

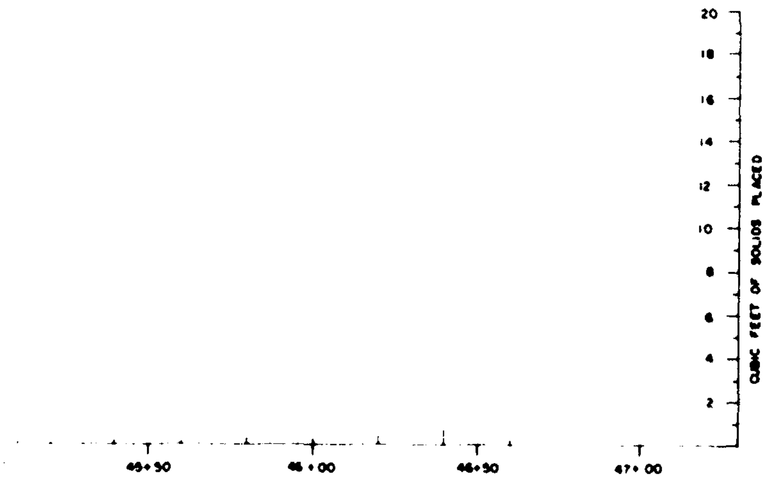
96

2


5
4
D
C
B
A
QPQT S T P T S T P T S T P T S T P T S T P T S T P T S T P T S T P T S T P T S T P TQSQTQPQTQSQTQPQT S TQPQT SQTQPQT S T P TQSQT
ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCH LINE J, PLATE 96
CONCRETE DAM - STATION 23 + 45.98 TO 42 + 55.48
20
18
16
14
12
10
8
6
4
2
STATION
42+50
43+00
43+50
44+00
44+50
45+00
45+50
ZONE 2
CUBIC FEET OF SOLIDS PLACED
MATCH LINE K PLATE 96
CONCRETE DAM - STATION 23 + 45.98 TO 42 + 55.48
ZONE 3
CUBIC FEET OF SOLIDS PLACED
MATCH LINE L, PLATE 96
CONCRETE DAM - STATION 23 + 45.98 TO 42 + 55.48







NOTES

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85
2. All grout holes on this plate are inclined in a 20° westerly direction
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1 • 0 - 20'
ZONE 2 • 20 - 50
ZONE 3 • 50 - 90

P • Primary grout hole
S • Secondary grout hole
T • Tertiary grout hole
Q • Quaternary grout hole

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FOUNDATION REPORT - EMBANKMENTS
GROUT TAKES FOR GROUT LINE "A"
GA & SC EARTH EMBANKMENTS
(SHEET 5 OF 5)
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

DES	SPECIFICATION NO	FILE NO	PLATE
F			97

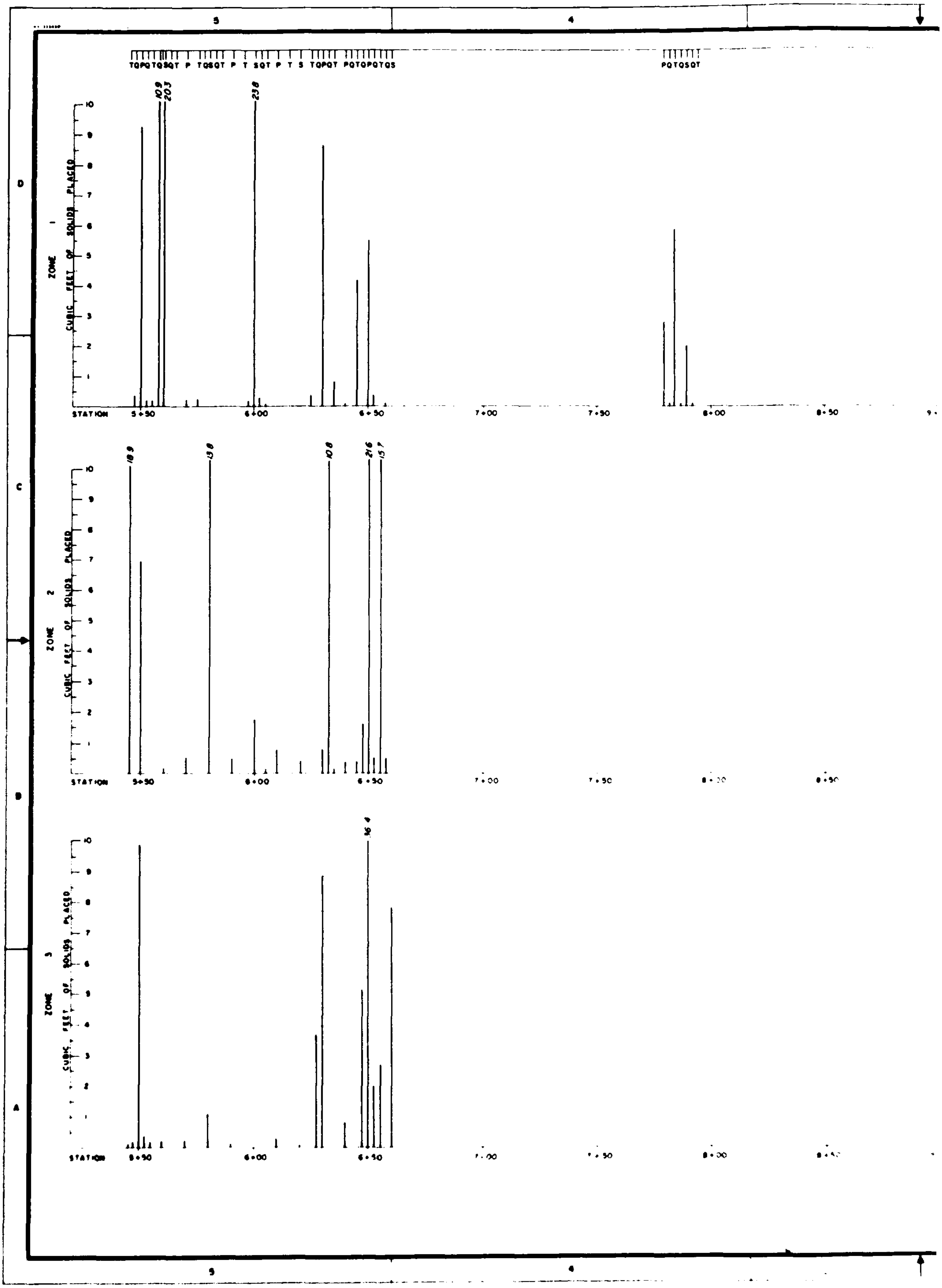
SCALE AS SHOWN | | SHEET

SYMBOL	ZONE	DESCRIPTION	DATE	BY

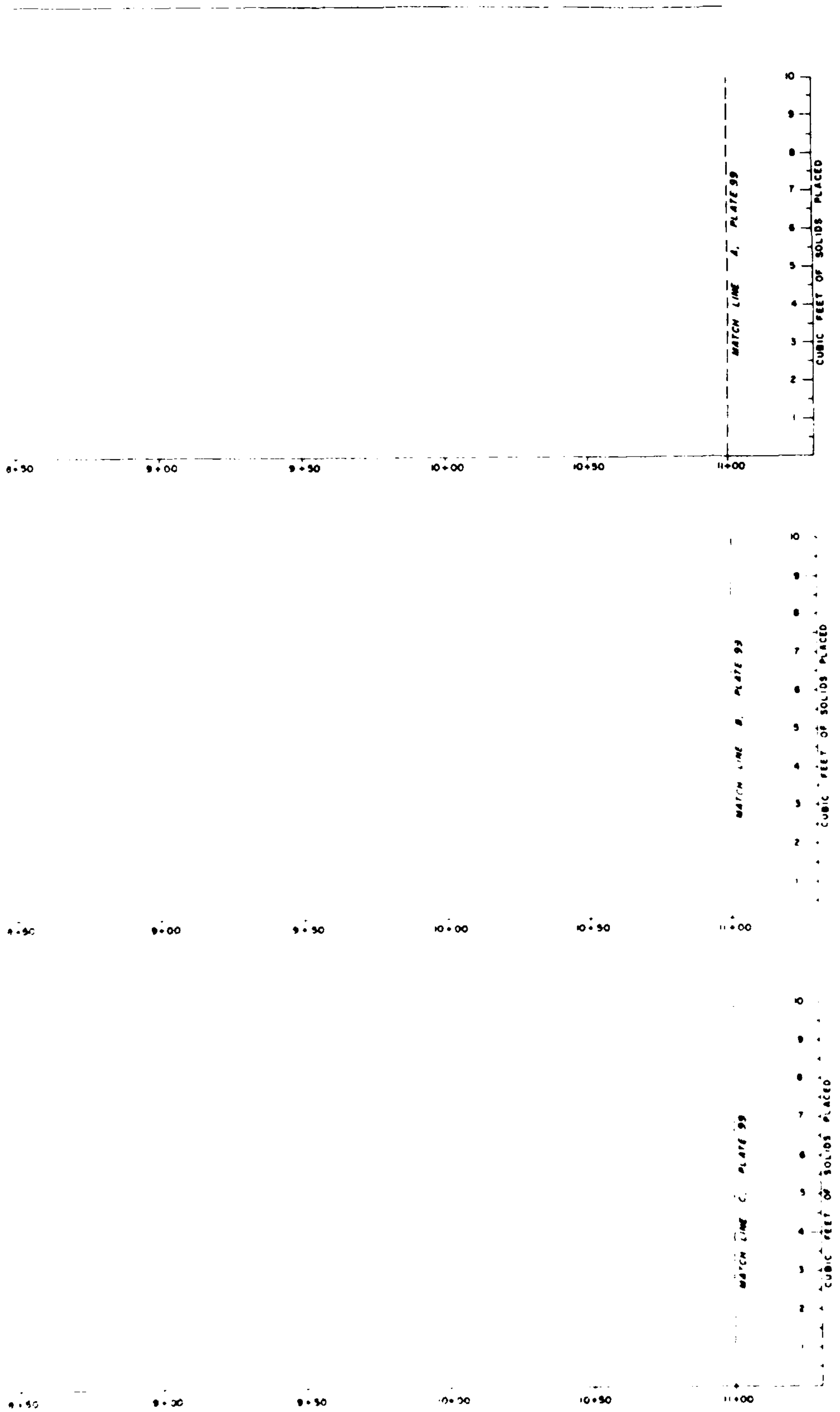
REVISIONS

97

2



TQPQTQSQT P TQSQT P T SQT P T S TQPQT PQTQPQTQS
PQTQSQT
ZONE 1
CUBIC FEET OF SOLIDS PLACED
10.9
20.3
23.8
STATION
5+50
6+00
6+50
7+00
7+50
8+00
8+50
ZONE 2
CUBIC FEET OF SOLIDS PLACED
18.9
13.8
10.8
21.6
15.7
ZONE 3
CUBIC FEET OF SOLIDS PLACED
16.4



NOTES:

1. For location of grout lines in plan view with respect to centerline, refer to Plates 84 and 85
2. All grout holes on this plate are inclined in a 20° westerly direction
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1 = 0-20
ZONE 2 = 20-50
ZONE 3 = 50-90

P = Primary grout hole
S = Secondary grout hole
T = Tertiary grout hole
Q = Quaternary grout hole

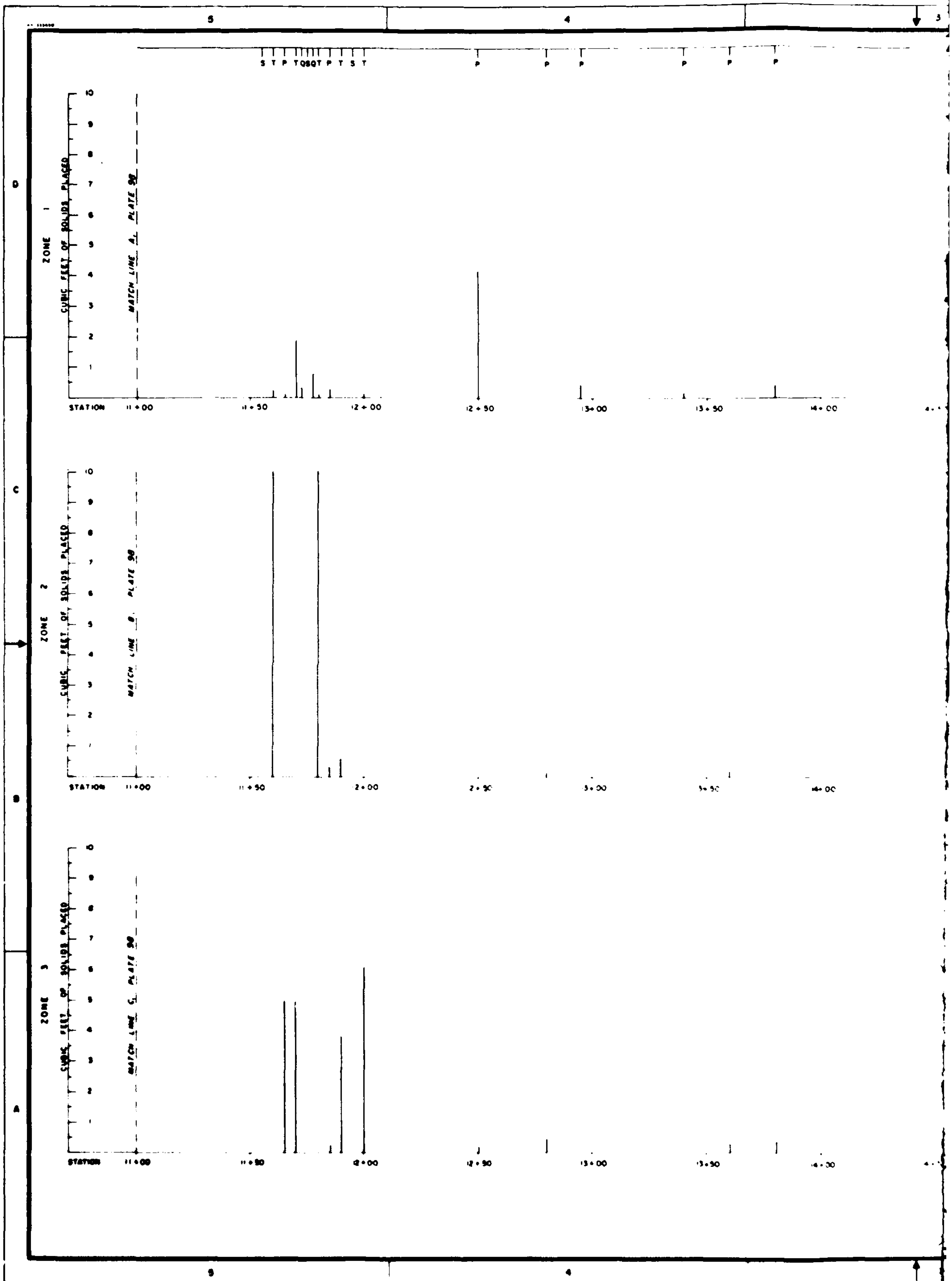
REVISIONS

U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

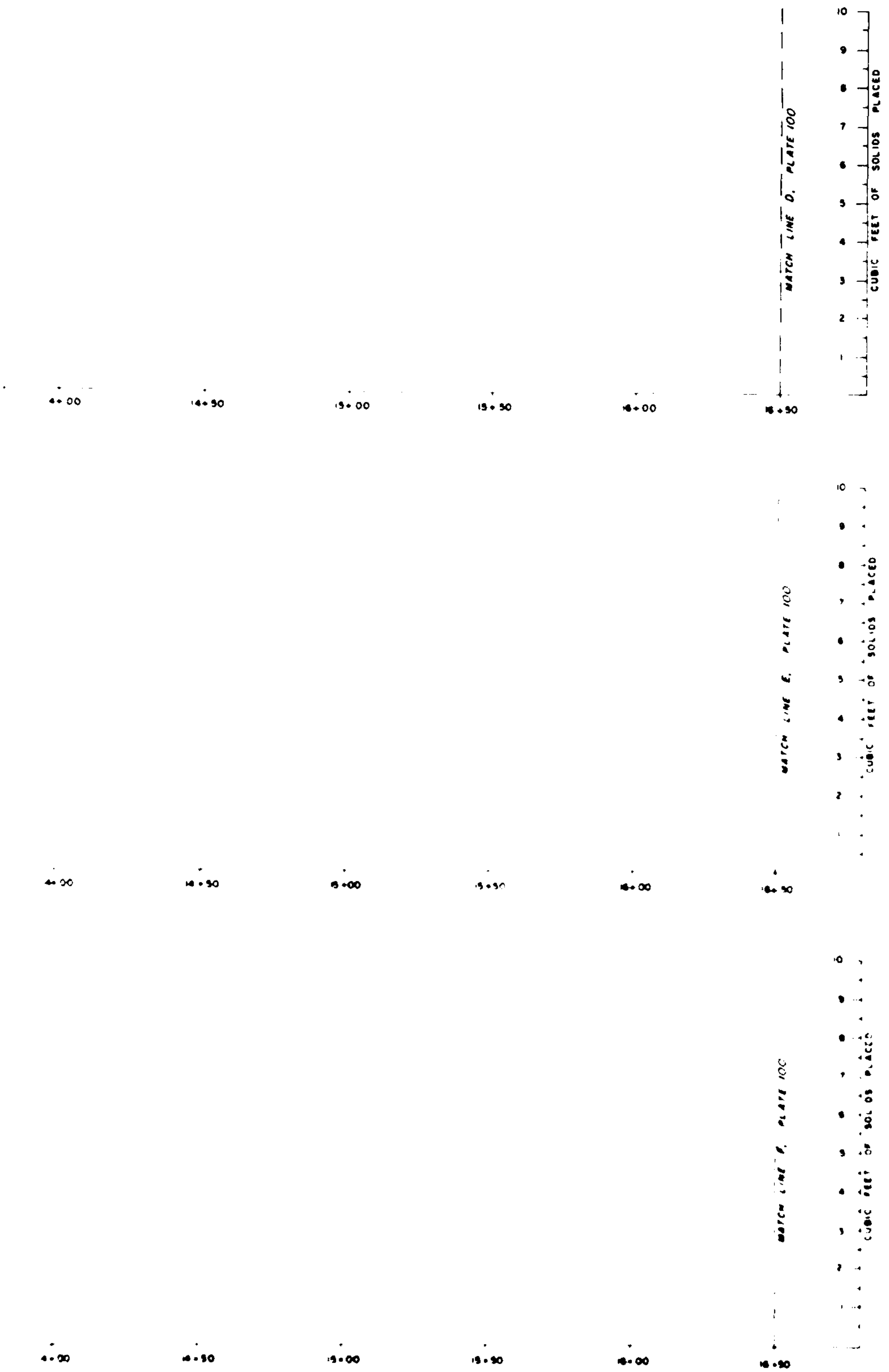
FOUNDATION REPORT – EMBANKMENTS
GROUT TAKES FOR GROUT LINE "B"
GA & SC EARTH EMBANKMENTS
(SHEET 1 OF 4)
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SCALE AS SHOWN

PLATE 98



ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCH LINE A, PLATE 30
ZONE 2
CUBIC FEET OF SOLIDS PLACED
MATCH LINE B, PLATE 30
ZONE 3
CUBIC FEET OF SOLIDS PLACED
MATCH LINE C, PLATE 30
STATION
11+00
11+50
12+00
12+50
13+00
13+50
14+00
S T P TQSQT P T S T
P



NOTES

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85
2. All grout holes on this plate are inclined in a 20° westerly direction
3. Field grouting records of individual grout holes will be maintained in the field of SASEN-GG for a limited time

ZONE 1 · 0 - 20
ZONE 2 · 20 - 50
ZONE 3 · 50 - 90

P · Primary grout hole
S · Secondary grout hole
T · Tertiary grout hole
Q · Quaternary grout hole

REVISIONS

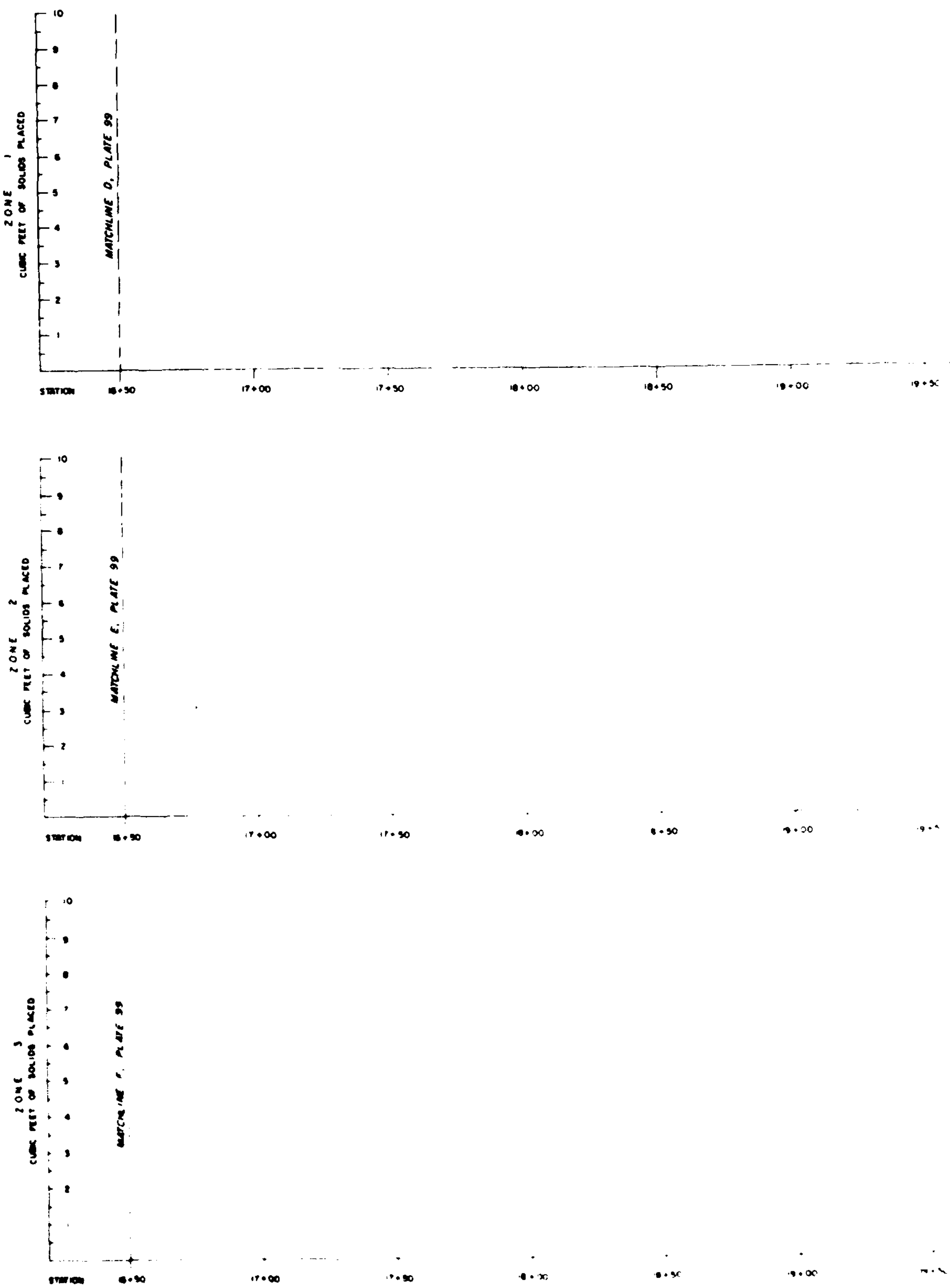
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT - EMBANKMENTS
GROUT TAKES FOR GROUT LINE "B"
GA & SC EARTH EMBANKMENTS
(SHEET 2 OF 4)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

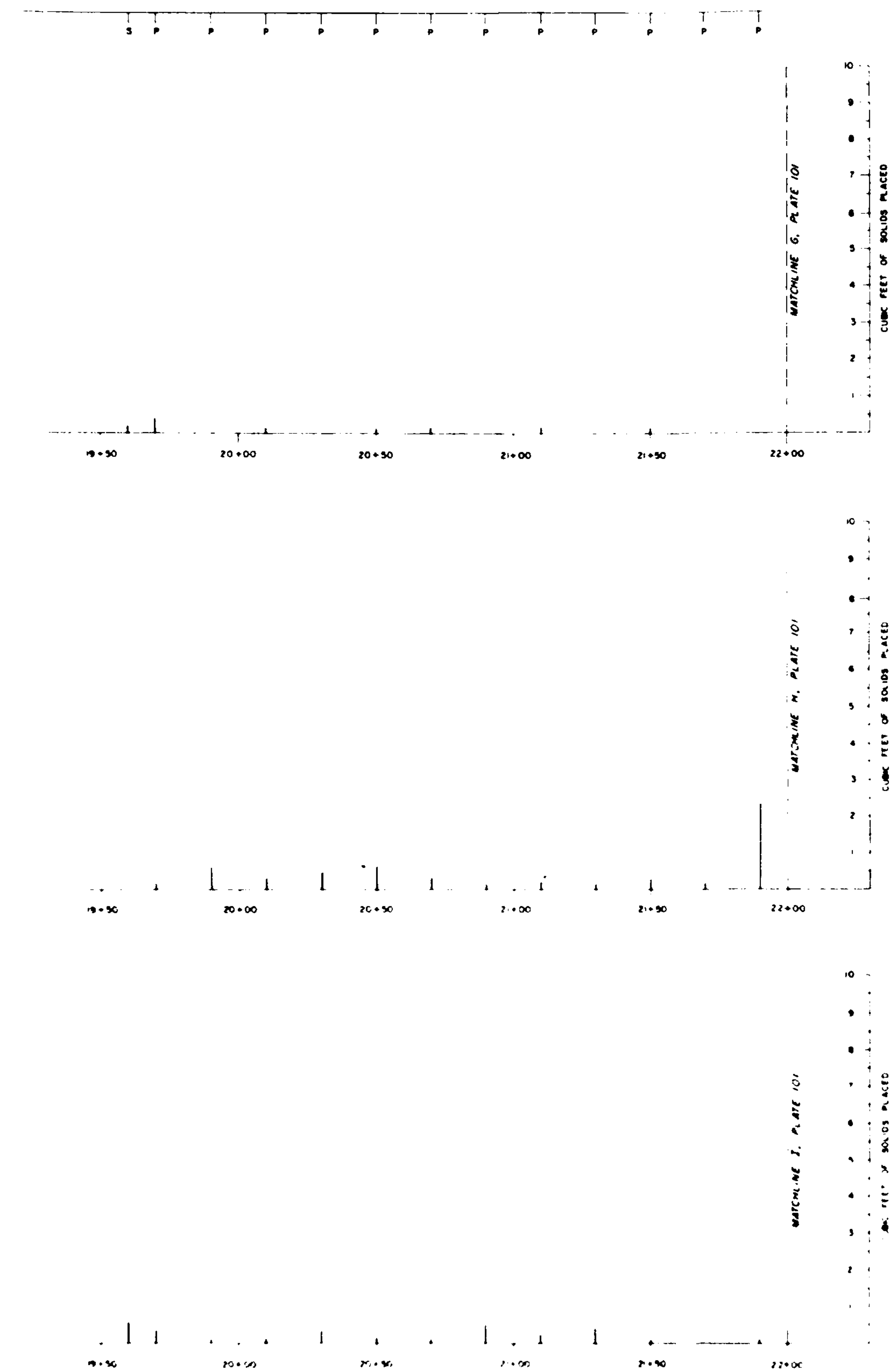
SCALE AS SHOWN

PLATE 99

99



ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCHLINE D, PLATE 99
STATION
16+50
17+00
17+50
18+00
18+50
19+00
ZONE 2
CUBIC FEET OF SOLIDS PLACED
MATCHLINE E, PLATE 99
ZONE 3
CUBIC FEET OF SOLIDS PLACED
PLATE 99



NOTES:

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85
2. All grout holes on this plate are inclined in a 20° westerly direction
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1 • 0 - 20
ZONE 2 • 20 - 50
ZONE 3 • 50 - 90'

P Primary grout hole
S Secondary grout hole
T Tertiary grout hole
Q Quarternary grout hole

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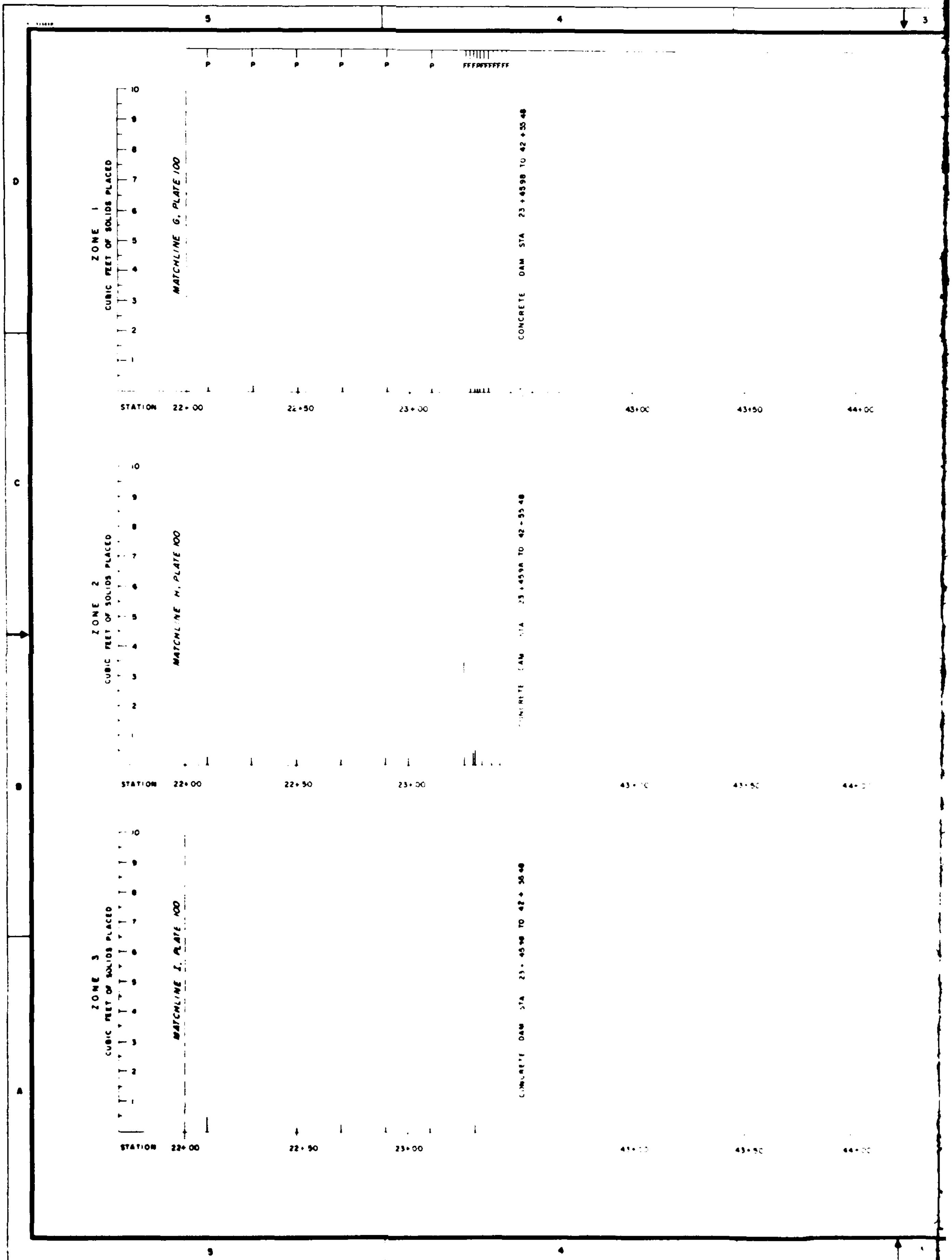
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT - EMBANKMENTS
GROUT TAKES FOR GROUT LINE "B"
GA & S.C. EARTH EMBANKMENTS
(SHEET 3 OF 4)
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

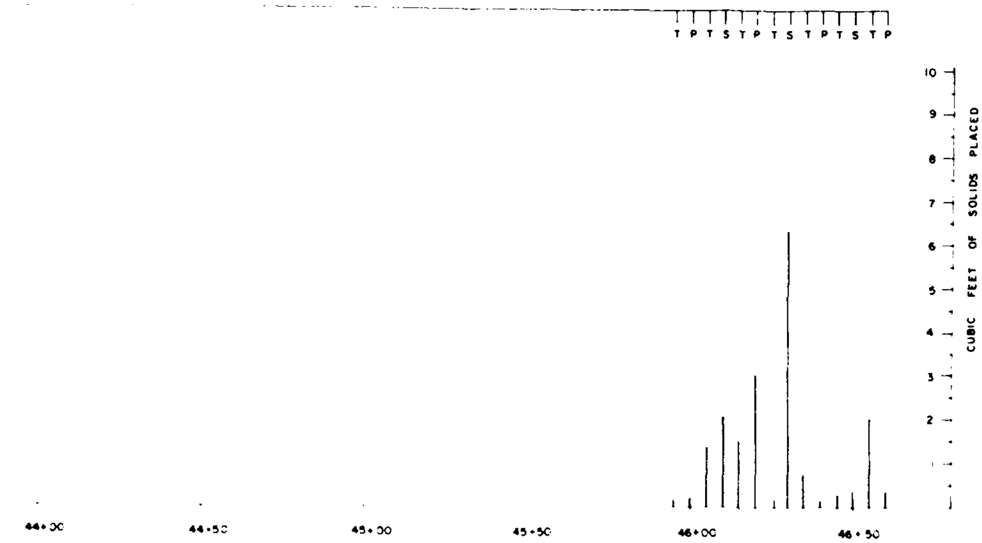
PLATE 100

SCALE AS SHOWN

2

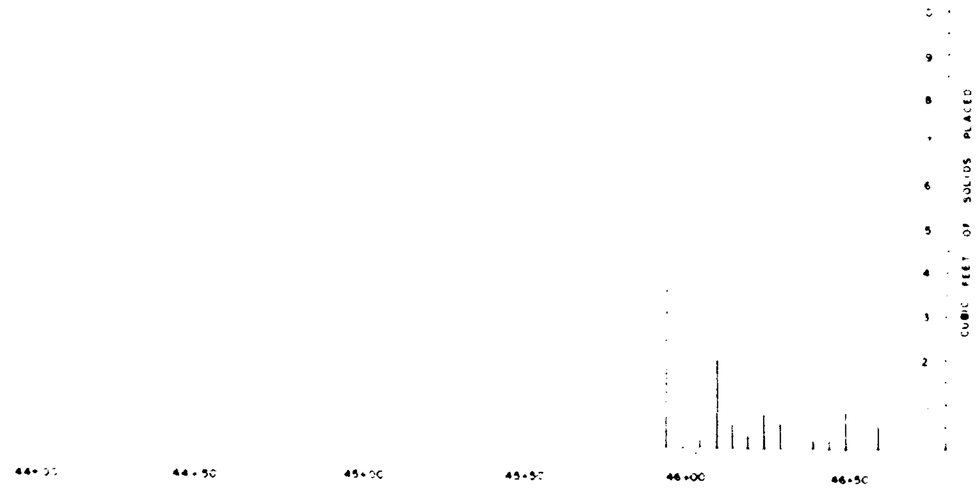


ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCHLINE G, PLATE 100
CONCRETE DAM STA 23+45.98 TO 42+35.48
STATION 22+00
22+50
23+00
43+00
43+50
44+00
ZONE 2
CUBIC FEET OF SOLIDS PLACED
MATCHLINE H, PLATE 100
STATION 22+00
22+50
23+00
ZONE 3
CUBIC FEET OF SOLIDS PLACED
MATCHLINE I, PLATE 100
CONCRETE DAM STA 23+45.98 TO 42+35.48
STATION 22+00
22+50
23+00

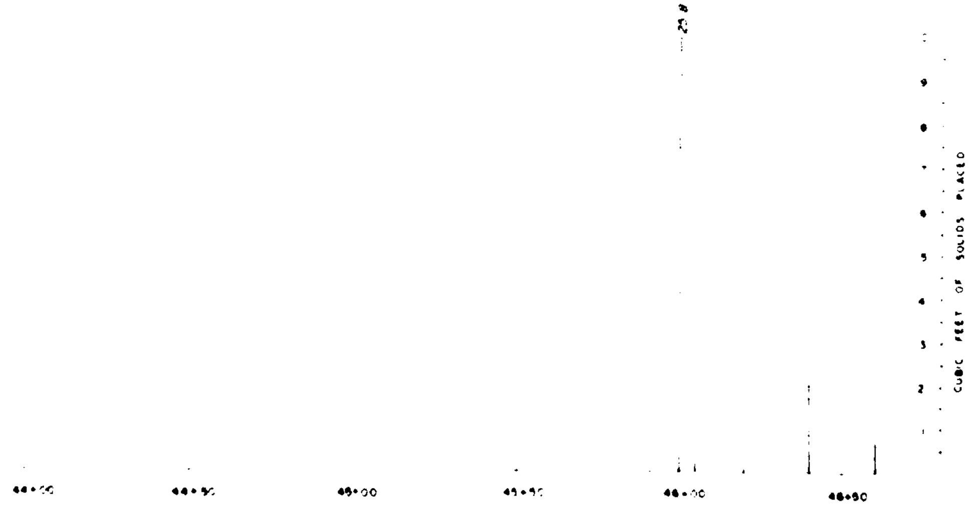


NOTES

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85.
2. All grout holes on this plat are inclined in a 20° westerly direction.
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time.



ZONE 1 - 0 - 20
ZONE 2 - 20 - 50
ZONE 3 - 50 - 90



P Primary grout hole
S Secondary grout hole
T Tertiary grout hole
Q Quarternary grout hole
F Fan grout hole

REVISIONS

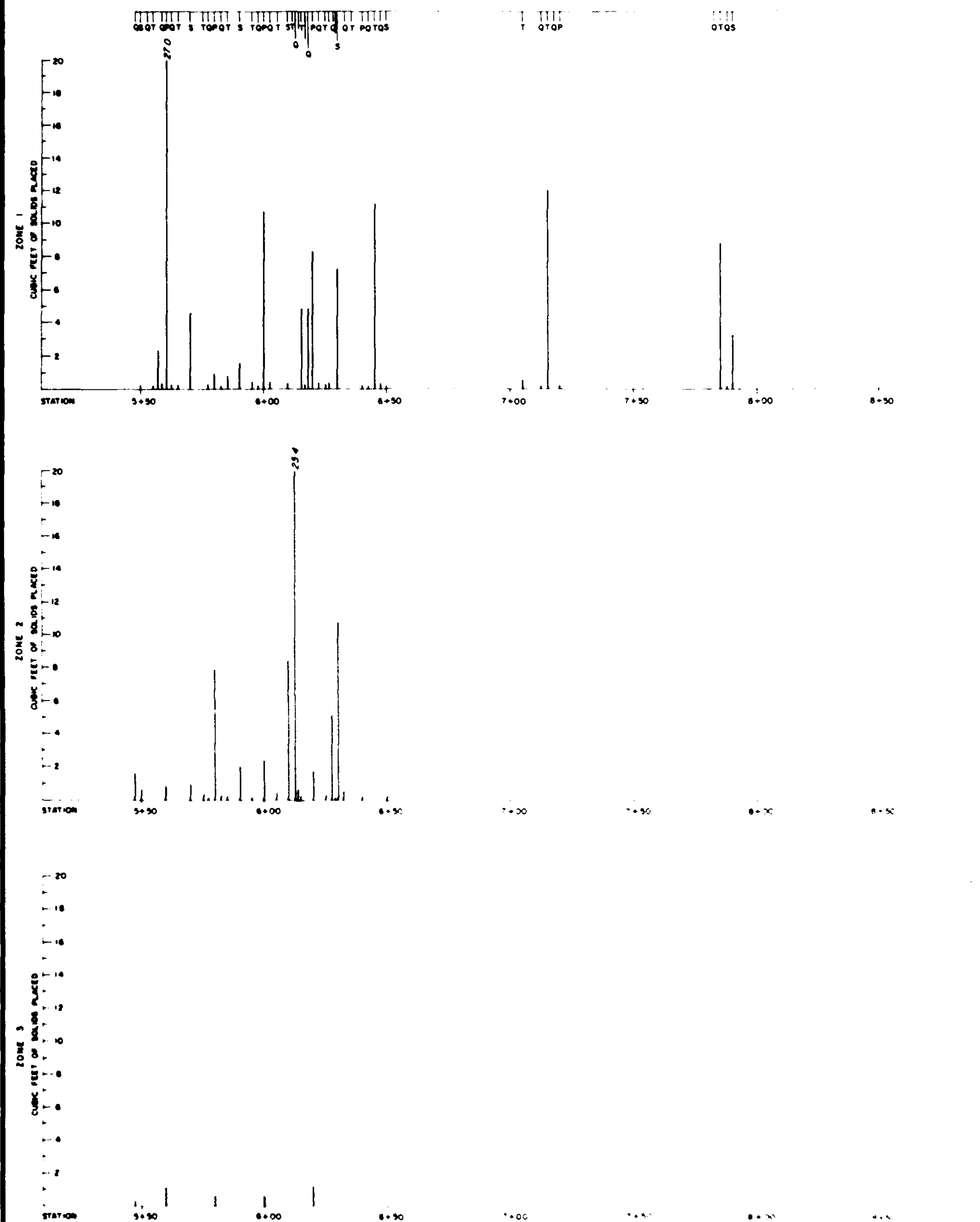
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT – EMBANKMENTS
GROUT TAKES FOR GROUT LINE "B"
GA. & S.C. EARTH EMBANKMENTS
(SHEET 4 OF 4)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

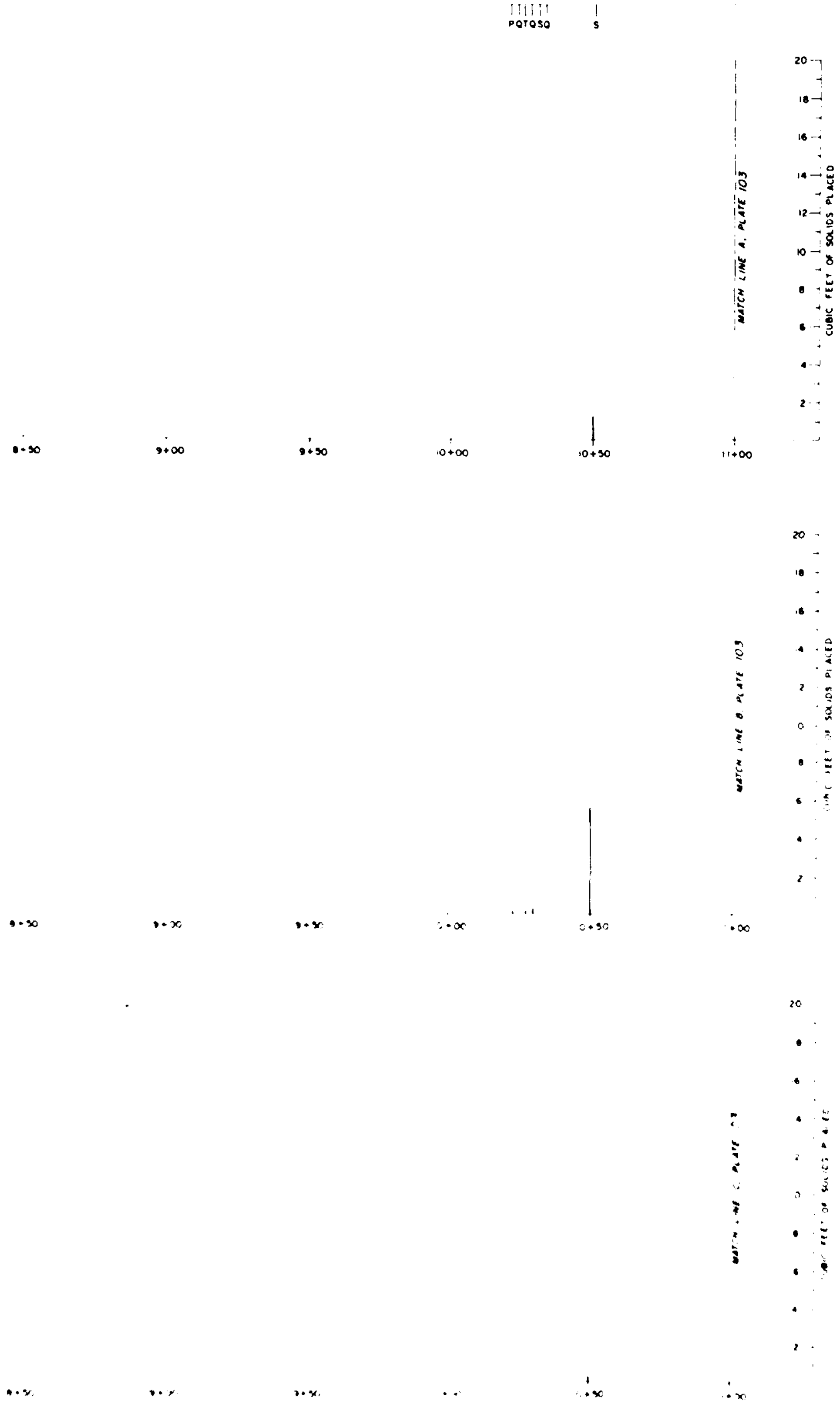
SIZE F
PLATE 101
SCALE AS SHOWN
SHEET

101

2



QSQT QPQT S TQPQT S TQPQT ST T PQT Q QT PQTQS
Q
Q
S
T QTQP
QTQS
270
ZONE 1
CUBIC FEET OF SOLIDS PLACED
20
18
16
14
12
10
8
6
4
2
STATION
5+50
6+00
6+50
7+00
7+50
8+00
8+50
254
ZONE 2
CUBIC FEET OF SOLIDS PLACED
STATION
ZONE 3
CUBIC FEET OF SOLIDS PLACED
STATION



NOTES

1. For location of grout lines in plan view with respect to centerline, refer to Plates 84 and 85

2. All grout holes on this plate are inclined in a 20° westerly direction

3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1 : 0 - 20
ZONE 2 : 20 - 50
ZONE 3 : 50 - 90

P - Primary grout hole
S - Secondary grout hole
T - Tertiary grout hole
Q - Quaternary grout hole

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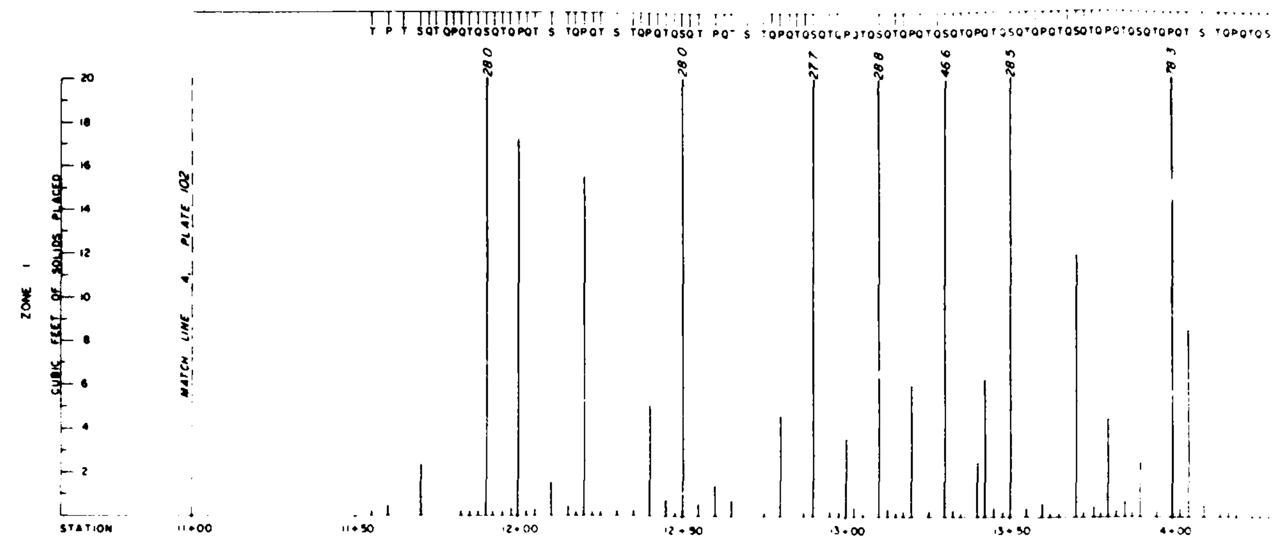
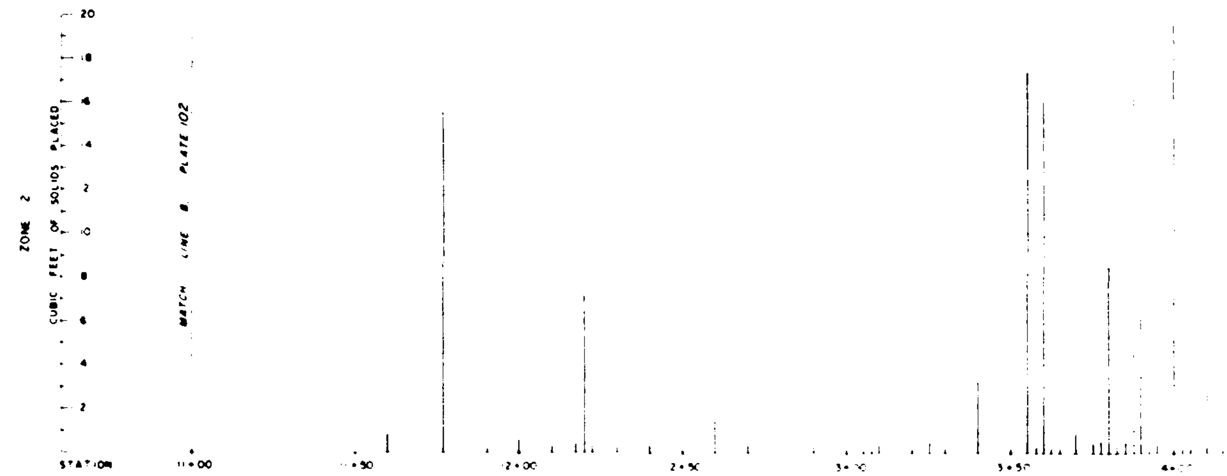
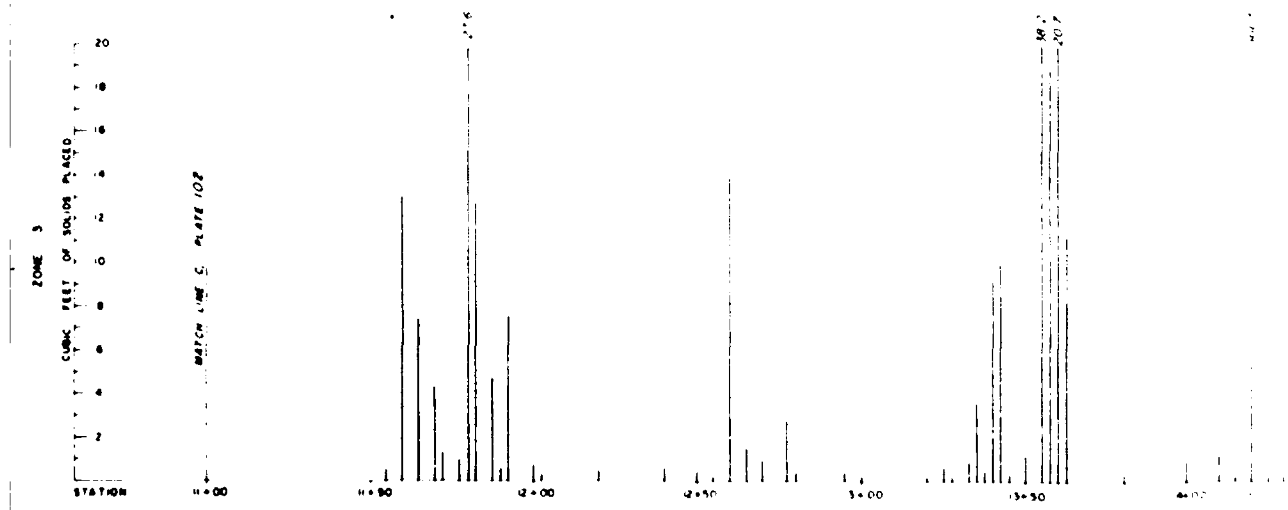
FOUNDATION REPORT EMBANKMENTS
GROUT TAKES FOR GROUT LINE "C"
GA & SC EARTH EMBANKMENTS
(SHEET 1 OF 4)
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

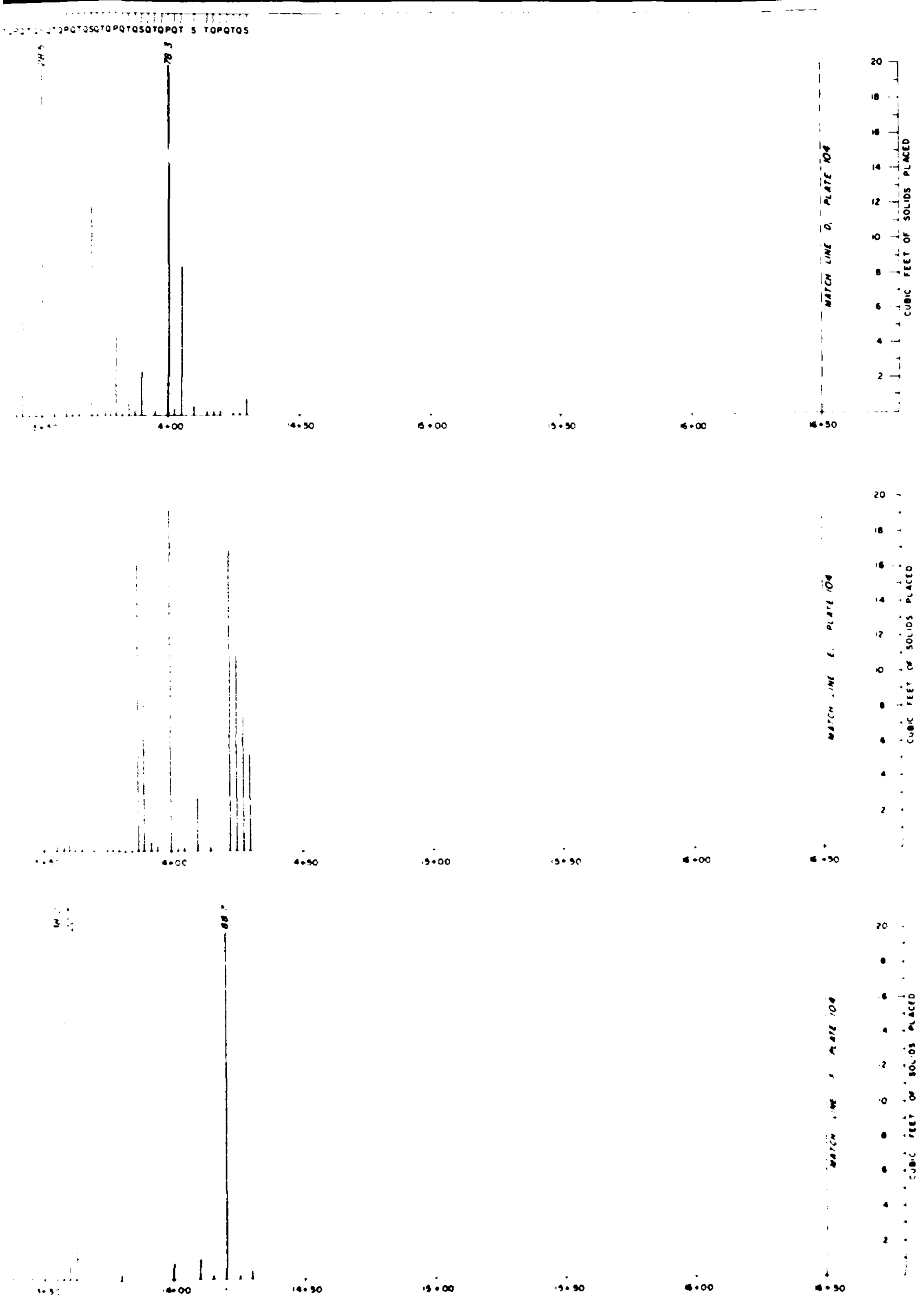
PLATE 102

102


ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCH LINE A, PLATE 102
STATION
11+00
11+50
12+00
12+50
13+00
13+50
14+00
28.0
28.0
27.7
28.8
46.6
28.5
78.3


ZONE 2
CUBIC FEET OF SOLIDS PLACED
MATCH LINE B, PLATE 102
STATION
11+00
12+00


ZONE 3
CUBIC FEET OF SOLIDS PLACED
MATCH LINE C, PLATE 102
STATION
11+00
11+50
12+00
12+50
13+00
13+50



NOTES

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85
2. All grout holes on this plate are inclined in a 20° westerly direction
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1-0-20
ZONE 2-20-50
ZONE 3-50-90

P - Primary grout hole
S - Secondary grout hole
T - Tertiary grout hole
Q - Quaternary grout hole

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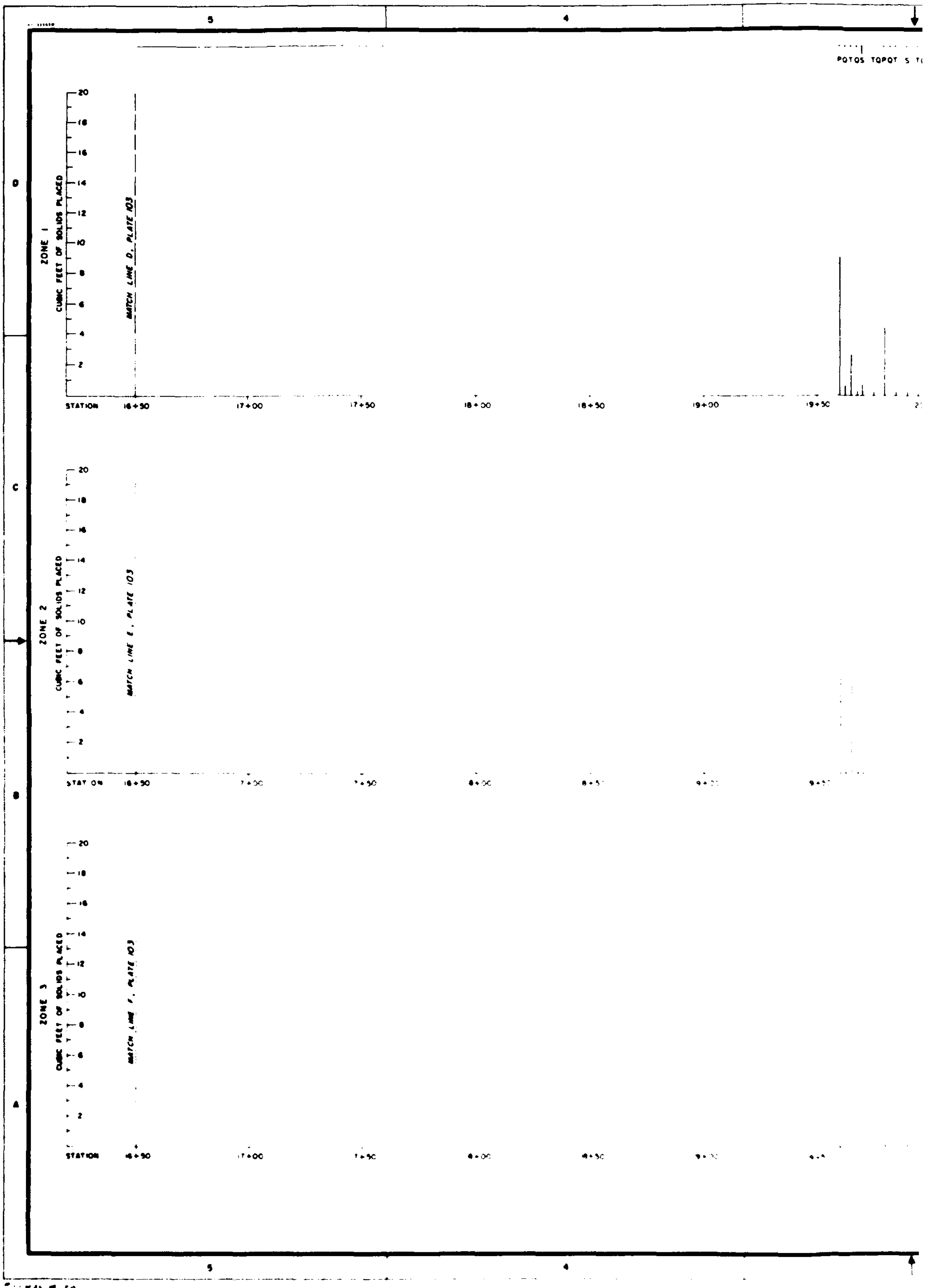
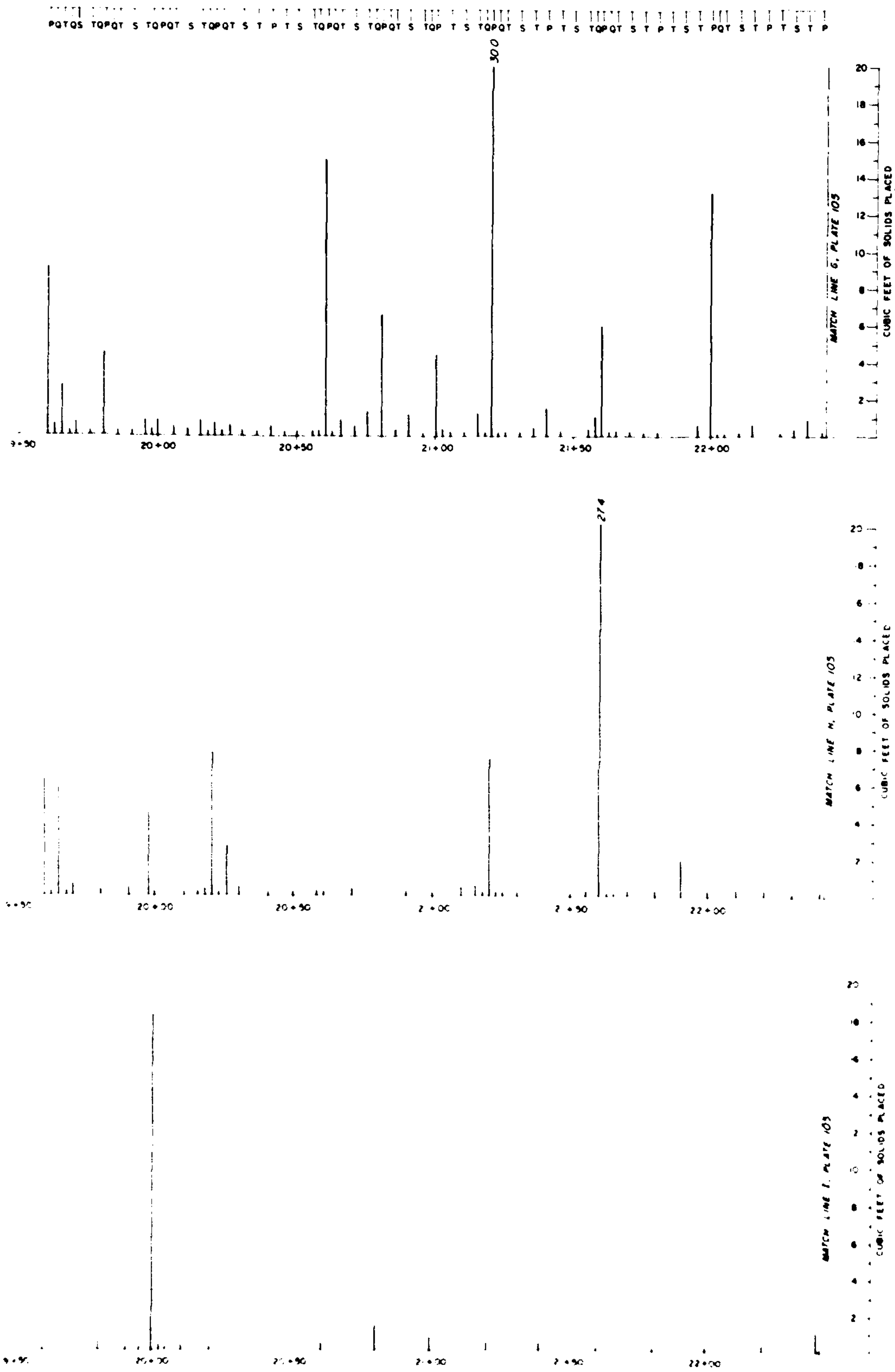
FOUNDATION REPORT - EMBANKMENTS
GROUT TAKES FOR GROUT LINE "C"
GA & SC EARTH EMBANKMENTS
(SHEET 2 OF 4)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE F
PLATE 103
SCALE AS SHOWN

REVISIONS

103


5
4
D
C
B
A
ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCH LINE D, PLATE 103
STATION
16+50
17+00
17+50
18+00
18+50
19+00
19+50
ZONE 2
CUBIC FEET OF SOLIDS PLACED
MATCH LINE E, PLATE 103
ZONE 3
CUBIC FEET OF SOLIDS PLACED
MATCH LINE F, PLATE 103
STATION
16+50
17+00



NOTES.

1 For location of grout lines in plan view with respect to centerline, refer to Plates 84 and 85

2 All grout holes on this plate are inclined in a 20° westerly direction

3 Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1 • 0 20
ZONE 2 • 20 50
ZONE 3 • 50 90

P • Primary grout hole
S • Secondary grout hole
T • Tertiary grout hole
Q • Quaternary grout hole

REVISIONS

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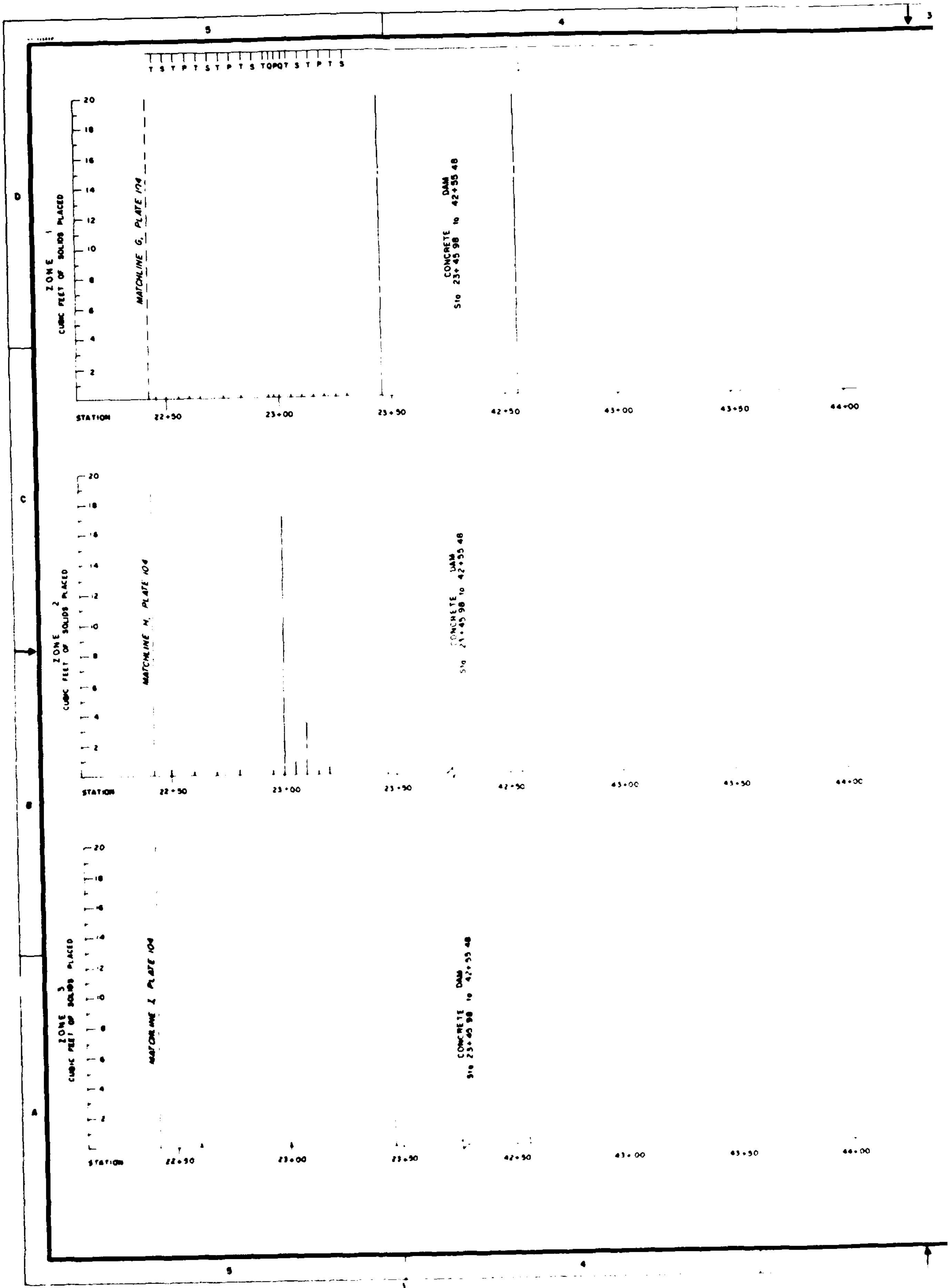
SAVANNAH, GEORGIA

FOUNDATION REPORT – EMBANKMENTS
GROUT TAKES FOR GROUT LINE "C"
GA & S.C. EARTH EMBANKMENTS
(SHEET 3 OF 4)
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

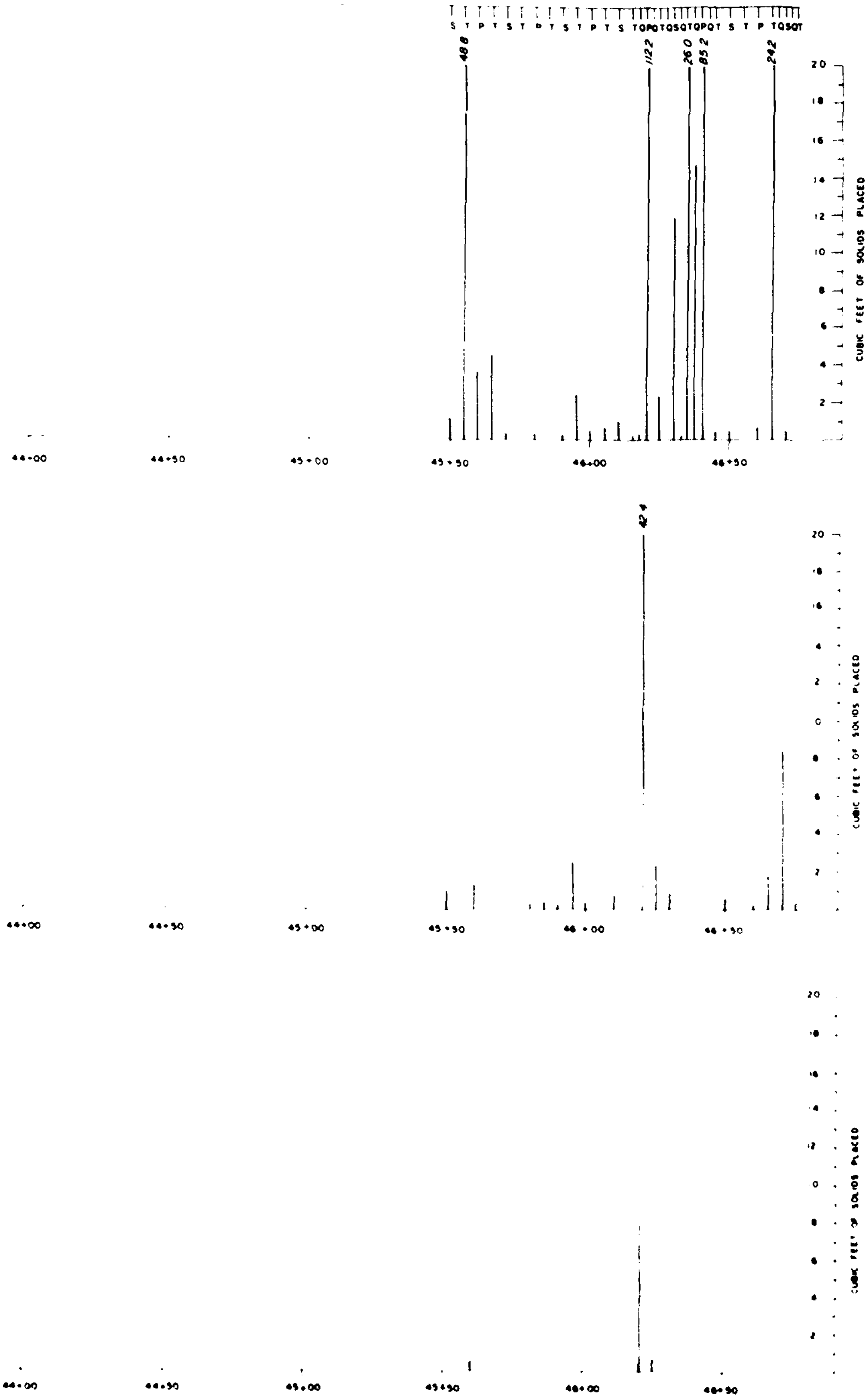
PLATE 104

SCALE AS SHOWN

104



ZONE 1
CUBIC FEET OF SOLIDS PLACED
MATCHLINE G, PLATE 104
CONCRETE DAM
Sta 23+45 98 to 42+55 48
STATION
22+50
23+00
23+50
42+50
43+00
43+50
44+00
ZONE 2
CUBIC FEET OF SOLIDS PLACED
MATCHLINE H, PLATE 104
CONCRETE DAM
Sta 23+45 98 to 42+55 48
STATION
22+50
23+00
23+50
42+50
43+00
43+50
44+00
ZONE 3
CUBIC FEET OF SOLIDS PLACED
MATCHLINE I, PLATE 104
CONCRETE DAM
Sta 23+45 98 to 42+55 48
STATION
22+50
23+00
23+50
42+50
43+00
43+50
44+00



NOTES:

1. For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85.
2. All grout holes on this plate are inclined in a 20° westerly direction
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1 - 0 - 20
ZONE 2 - 20 - 50
ZONE 3 - 50 - 90

P Primary grout hole
S Secondary grout hole
T Tertiary grout hole
Q Quarternary grout hole

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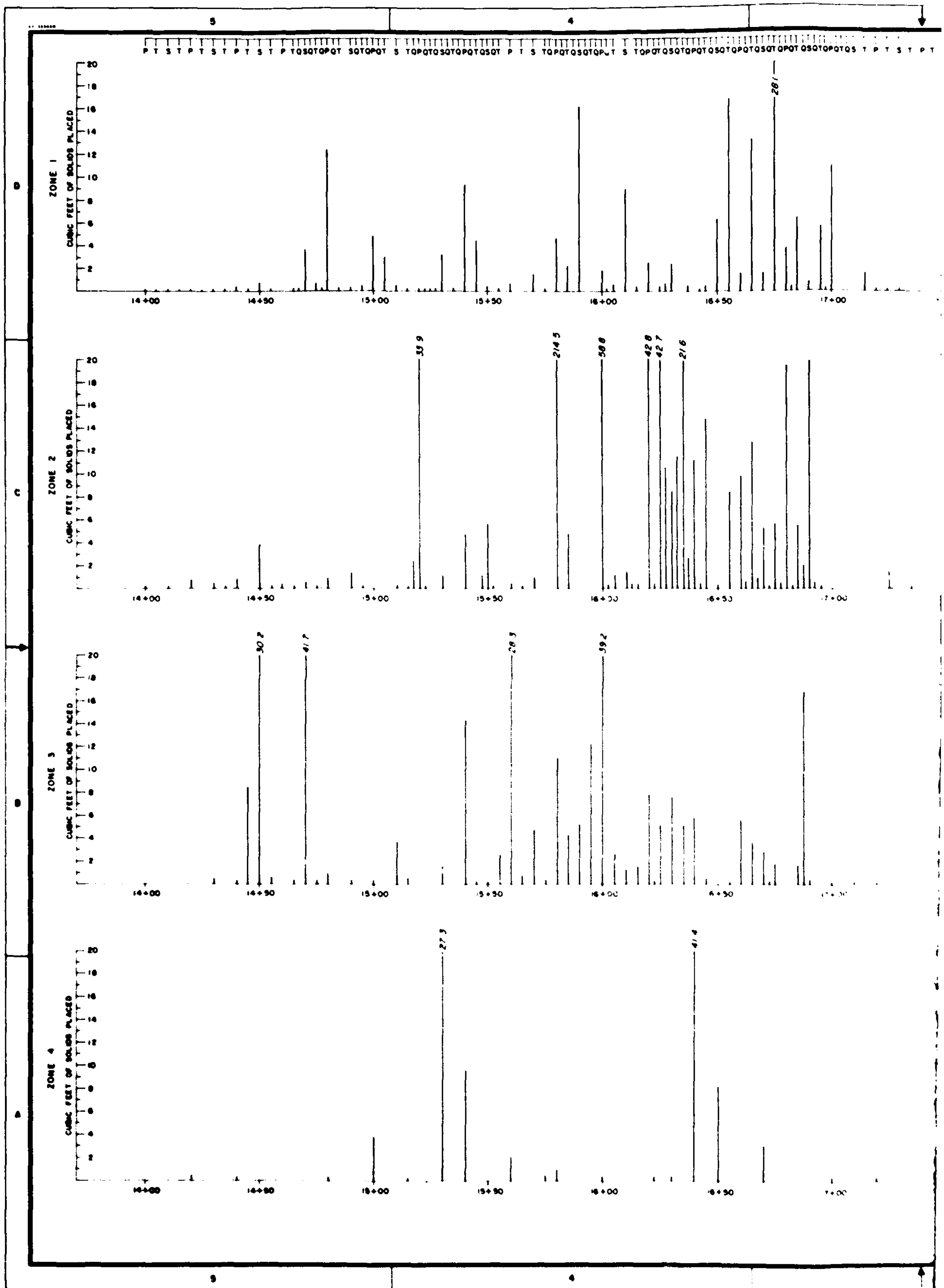
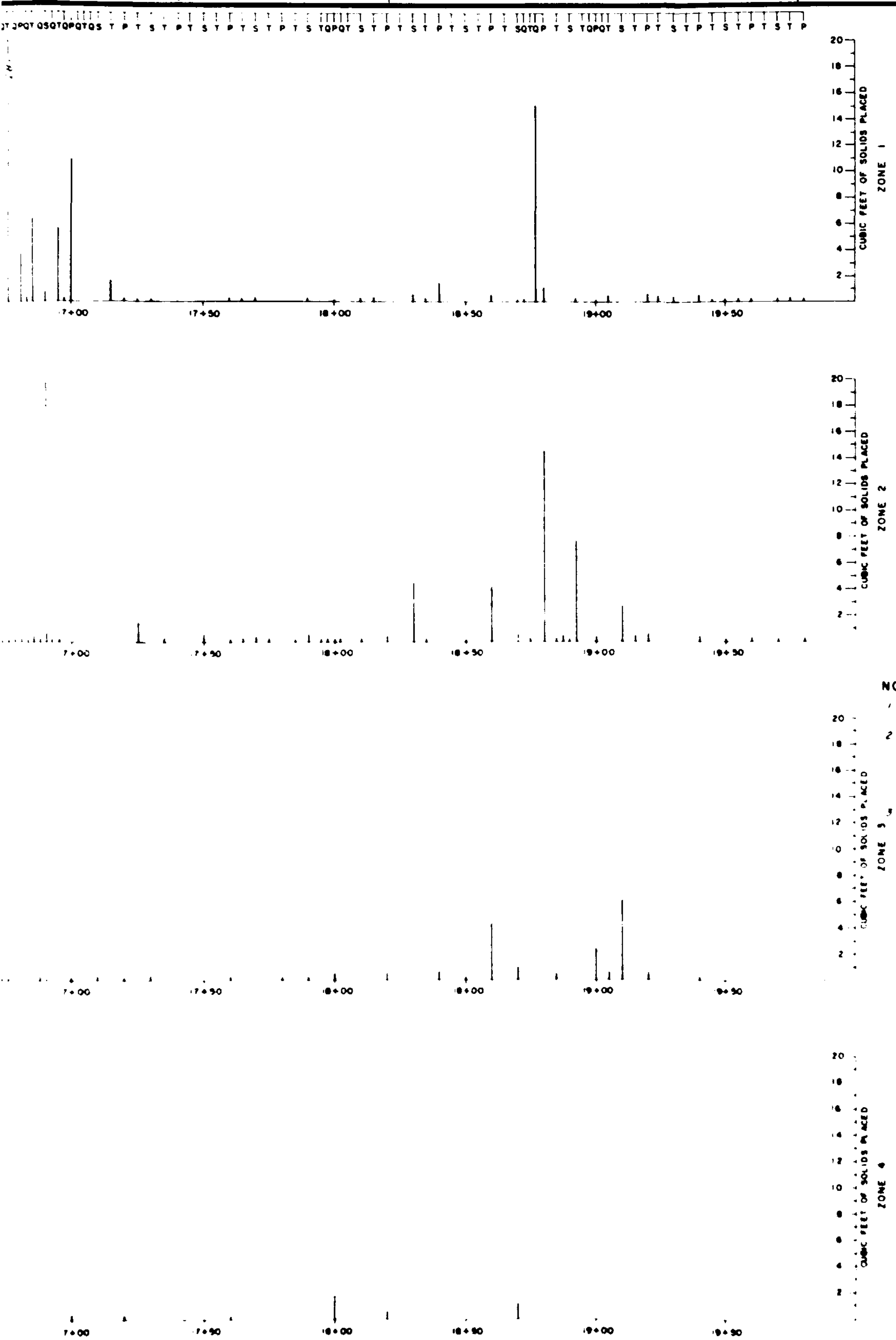
FOUNDATION REPORT - EMBANKMENTS
GROUT TAKES FOR GROUT LINE "C"
GA & SC EARTH EMBANKMENTS
(SHEET 4 OF 4)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 105

REVISIONS

SCALE AS SHOWN


5
4
ZONE 1
CUBIC FEET OF SOLIDS PLACED
28.1
14+00
14+50
15+00
15+50
16+00
16+50
17+00
ZONE 2
CUBIC FEET OF SOLIDS PLACED
33.9
214.5
58.8
42.8
42.7
21.6
ZONE 3
CUBIC FEET OF SOLIDS PLACED
30.2
41.7
28.3
39.2
ZONE 4
CUBIC FEET OF SOLIDS PLACED
27.3
41.4
D
C
B
A



NOTES

1. For location of grout line in plan view with respect to center-line refer to Plates 84 and 85
2. All grout holes located between station 14+00 and 16+20 are inclined in a 20° westerly direction. Grout holes from station 16+40 to 19+85 are inclined in a 20° easterly direction. Refer to Plate 89 for inclination of holes located from station 16+22.5 to 16+37.5
3. Field grouting records of individual grout holes will be maintained in the files of SASEN-GG for a limited time

ZONE 1 - 0 - 20
ZONE 2 - 20 - 50
ZONE 3 - 50 - 90
ZONE 4 - 90 - 120

P Primary grout hole
S Secondary grout hole
T Tertiary grout hole
Q Quarternary grout hole

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
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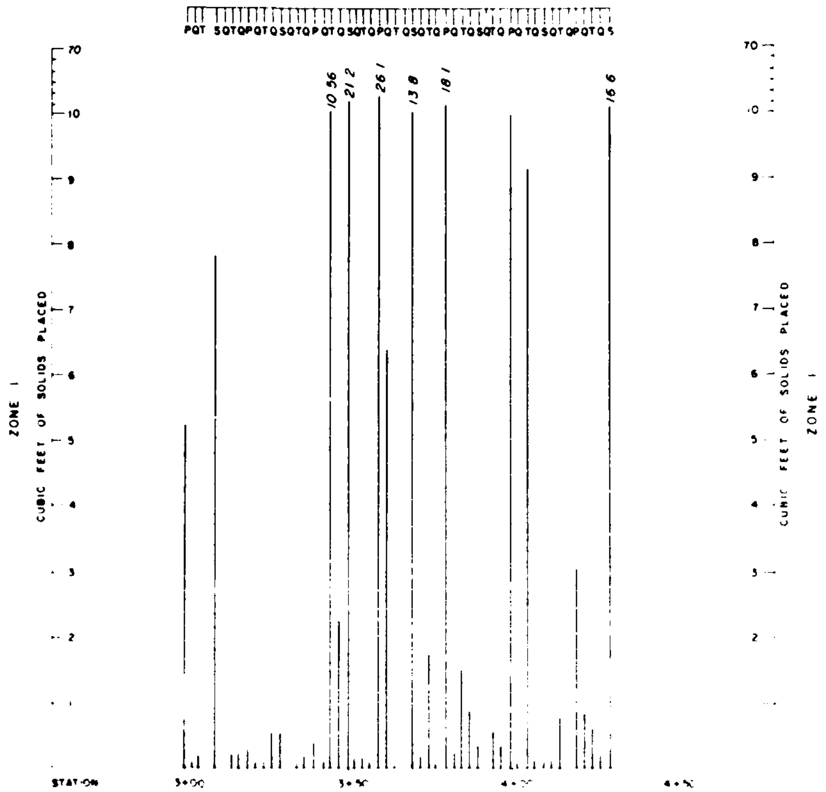
FOUNDATION REPORT
EMBANKMENTS
GROUT TAKES FOR THE DIVERSION-CHANNEL LINE, GA & S.C EARTH EMBANKMENTS
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 108

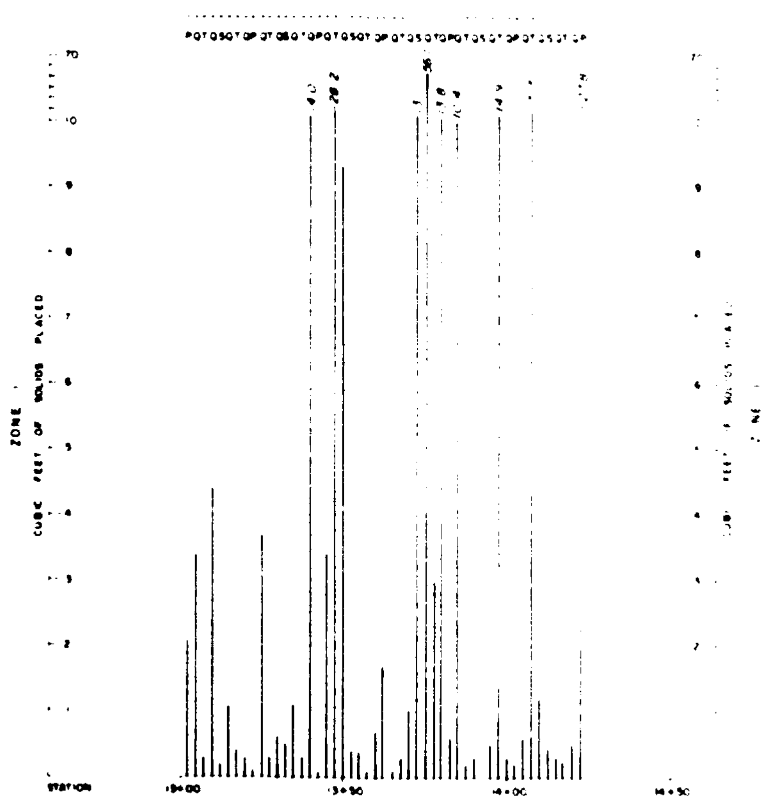
SCALE AS SHOWN

REVISIONS

5 4

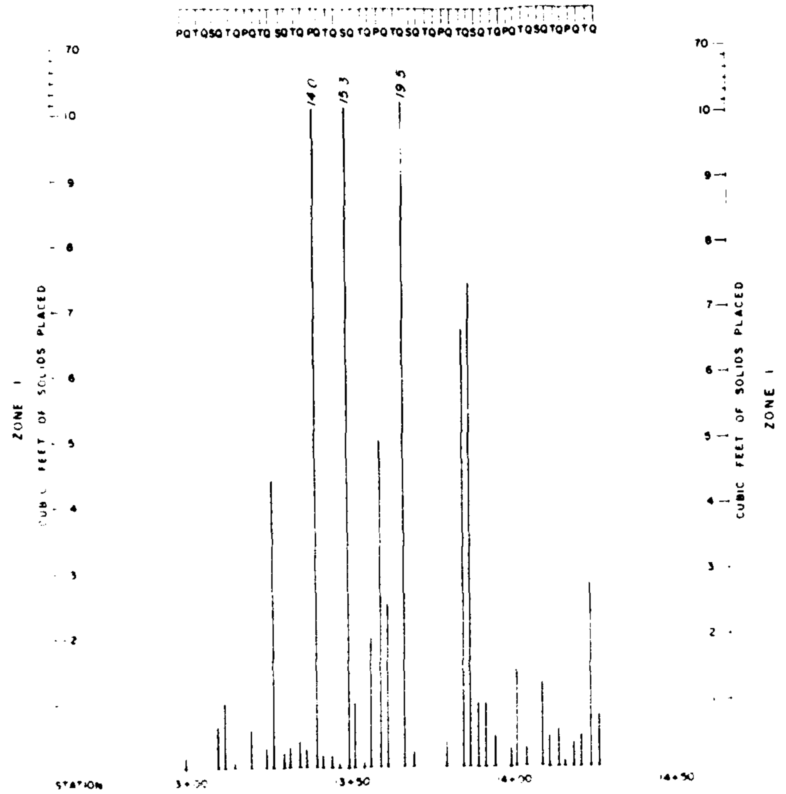


GROUT LINE D

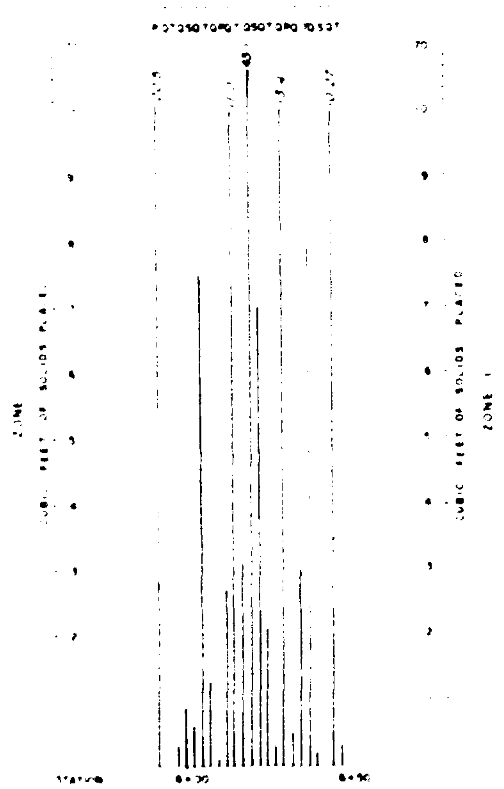


GROUT LINE F

5 4



GROUT LINE E



PERIMETER GROUT LINE

NOTES

For location of grout line in plan view with respect to centerline, refer to Plates 84 and 85

2. All grout holes on this plate are inclined in a 20° westerly direction

3. Field grouting records of individual grout holes will be maintained in the files of SASEN-?? for a limited time

ZONE 1 + 0 20

P - Primary grout hole
S - Secondary grout hole
T - Tertiary grout hole
Q - Quaternary grout hole

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FOUNDATION REPORT
EMBANKMENTS

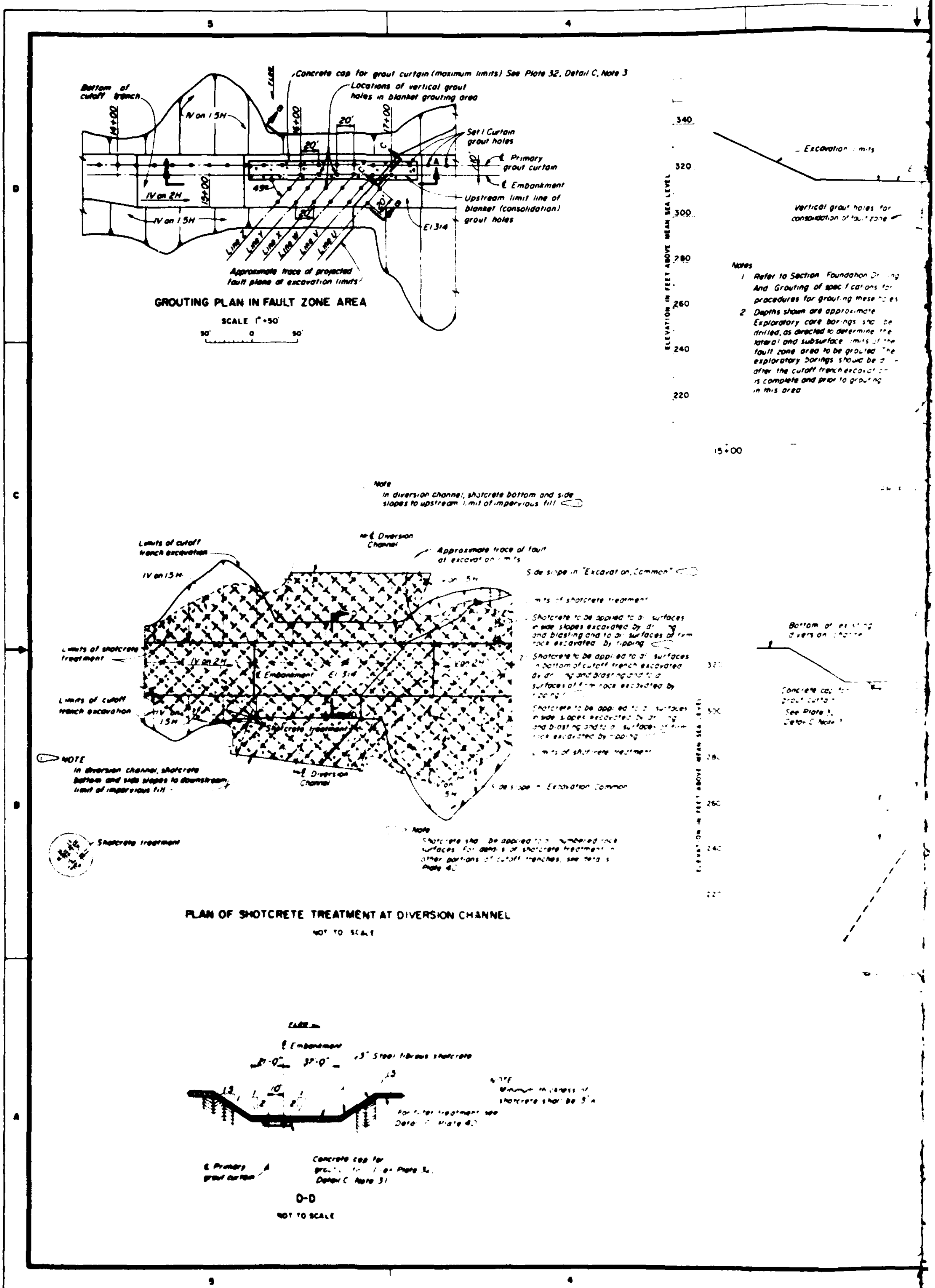
GROUT TAKES FOR D, E, F & PERIMETER GROUT LINES - GA. EARTH EMBANKMENT

RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SCALE AS SHOWN

PLATE 107

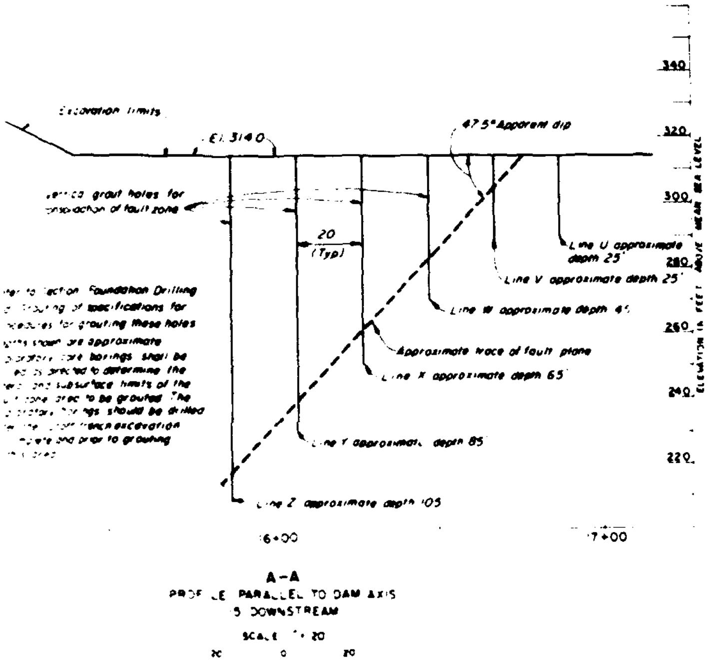
REVISIONS



Concrete cap for grout curtain (maximum limits) See Plate 32, Detail C, Note 3
Locations of vertical grout holes in blanket grouting area
Bottom of cutoff trench
14+00
1V on 1.5H
16+00
17+00
20'
Set I Curtain grout holes
Primary grout curtain
℄ Embankment
1V on 2H
15+00
45°
Upstream limit line of blanket (consolidation) grout holes
1V on 1.5H
El 314
Line Z
Line Y
Line X
Line W
Line V
Line U
Approximate trace of projected fault plane at excavation limits
GROUTING PLAN IN FAULT ZONE AREA
SCALE 1"=50'
50'
0
50'
340
320
300
280
260
240
220
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
Excavation limits
Vertical grout holes for consolidation of fault zone
Notes
1 Refer to Section Foundation Drilling And Grouting of specifications for procedures for grouting these holes
2 Depths shown are approximate. Exploratory core borings should be drilled, as directed to determine the lateral and subsurface limits of the fault zone area to be grouted. The exploratory borings should be drilled after the cutoff trench excavation is complete and prior to grouting in this area
15+00
Note
In diversion channel, shotcrete bottom and side slopes to upstream limit of impervious fill
Limits of cutoff trench excavation
℄ Diversion Channel
Approximate trace of fault at excavation limits
Side slope in "Excavation, Common"
1V on 1.5H
Limits of shotcrete treatment
Shotcrete to be applied to all surfaces inside slopes excavated by drilling and blasting and to all surfaces of firm rock excavated by ripping
Shotcrete to be applied to all surfaces in bottom of cutoff trench excavated by drilling and blasting and to all surfaces of firm rock excavated by ripping
Shotcrete to be applied to all surfaces inside slopes excavated by drilling and blasting and to all surfaces of firm rock excavated by ripping
Limits of shotcrete treatment
Limits of shotcrete treatment
1V on 2H
℄ Embankment
El 314
Limits of cutoff trench excavation
1V on 1.5H
Shotcrete treatment
NOTE
In diversion channel, shotcrete bottom and side slopes to downstream limit of impervious fill
℄ Diversion Channel
Side slope in "Excavation, Common"
Note
Shotcrete shall be applied to all numbered rock surfaces. For details of shotcrete treatment in other portions of cutoff trenches, see details Plate 42
Shotcrete treatment
Bottom of existing diversion channel
Concrete cap for grout curtain See Plate 32, Detail C, Note 3
PLAN OF SHOTCRETE TREATMENT AT DIVERSION CHANNEL
NOT TO SCALE
℄ Embankment
21'-0"
37'-0"
3" Steel fibrous shotcrete
1.5
10'
NOTE
Minimum thickness of shotcrete shall be 3" in
For filter treatment see Detail Plate 43
℄ Primary grout curtain
Concrete cap for grout curtain See Plate 32, Detail C, Note 3
D-D
NOT TO SCALE
5
4
D
C
B
A

NOTE:

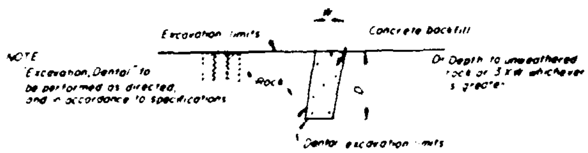
1. This report does not include drawings that show the "as built" limits of shotcrete and dental concrete placement, refer to Embankment Criteria and Performance Report.



A–A

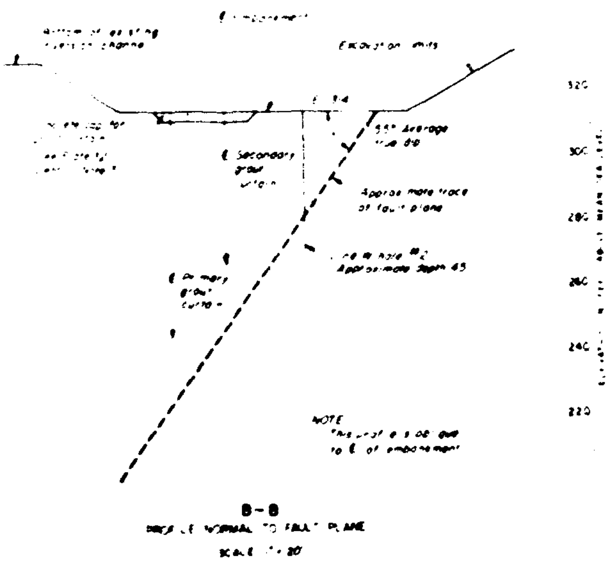
PROFILE PARALLEL TO DAM AXIS 15 DOWNSTREAM

SCALE 1"=20



DETAIL OF DENTAL EXCAVATION AND CONCRETE TREATMENT (TYPICAL)

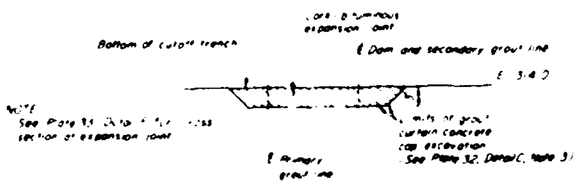
NOT TO SCALE



B–B

PROFILE NORMAL TO FAULT PLANE

SCALE 1"=20



C–C

PROFILE OF EXPANSION JOINT IN CONCRETE CAP AT FAULT ZONE

NOT TO SCALE

NOTE

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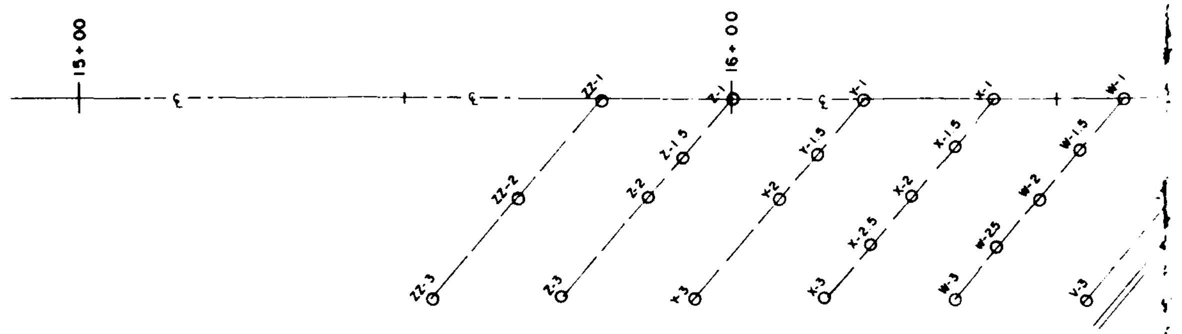
FOUNDATION REPORT
EMBANKMENTS
GROUTING OF GEOLOGIC FAULT AND SPECIAL TREATMENT (AS-DESIGNED)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

108

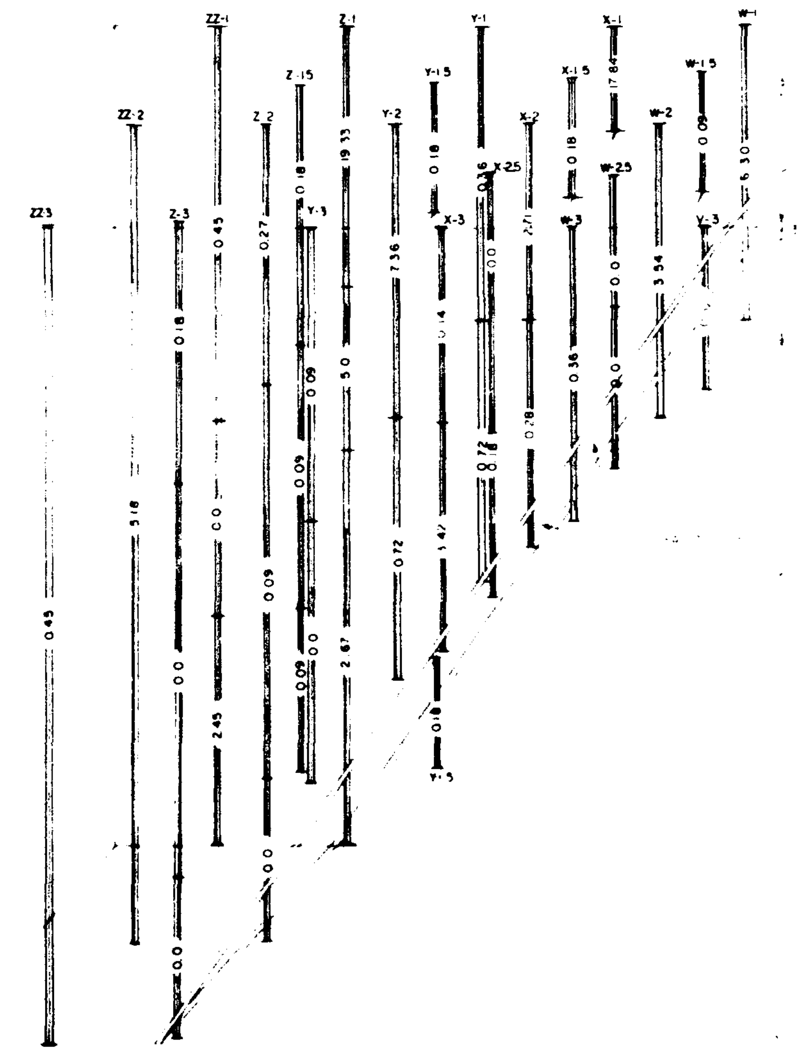
REVISIONS

2



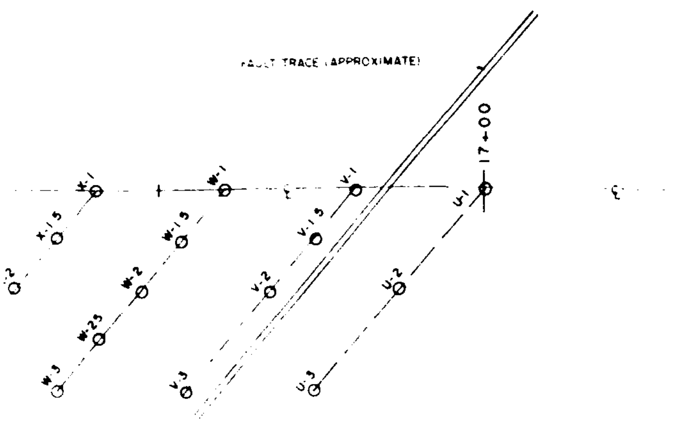
AS-BUILT CONSOLIDATION GROUTING (PLAN VIEW)

SCALE: 1" = 10'

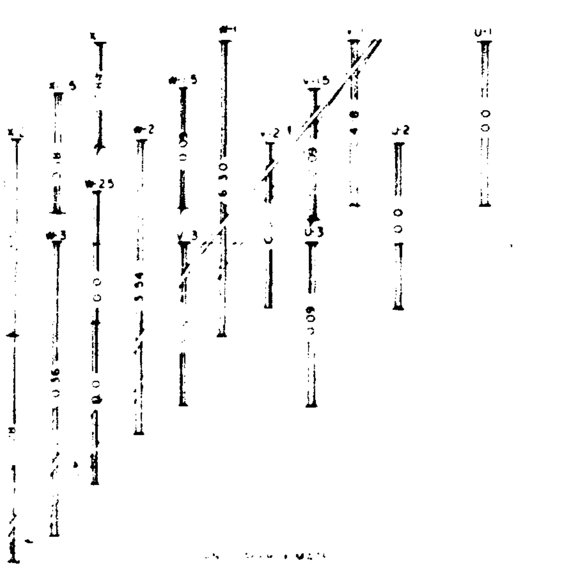


AS-BUILT CONSOLIDATION GROUTING (DIAGRAMMATIC P

SCALE 1" = 10'



GROUTING (PLAN VIEW)



N GROUTING (DIAGRAMMATIC PROFILE)

NOTES:

1. FOR A VERBAL DESCRIPTION OF CONSOLIDATION GROUTING IN THE DIVERSION CHANNEL FAULT TRACE AREA, REFER TO SECTION 7.4-EE OF TEXT
2. ALL GROUTING DONE BY THE "STOP"-GROUTING METHOD
3. SEE PLATE 108 FOR CONTRACT DESIGN OF THIS SPECIALIZED TREATMENT
4. GROUT TAKES SHOWN ARE IN CUBIC FEET TAKEN PER INTERVAL OF GROUT HOLE
5. ORIENTATION OF FAULT PLANE IS THEORETICALLY PROJECTED BASED ON BORING DATA AND OTHER FIELD OBSERVATIONS

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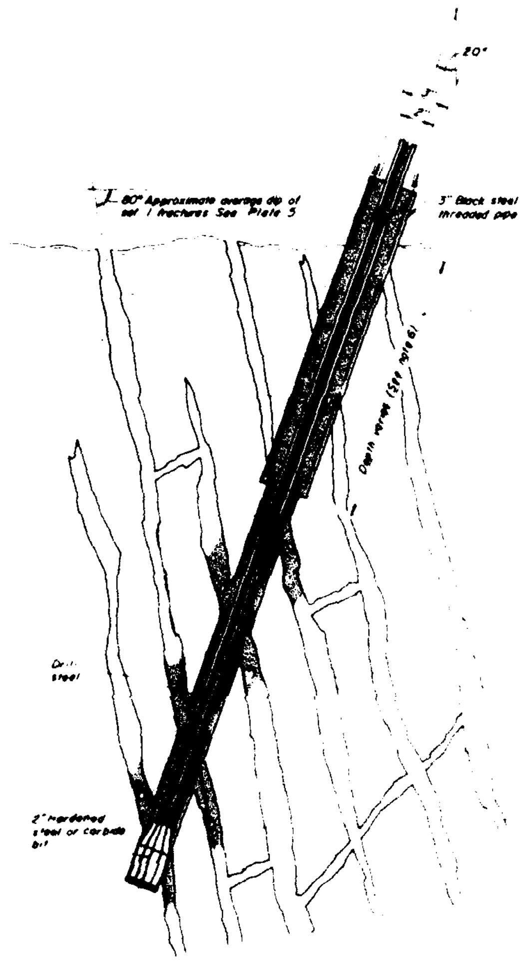
FOUNDATION REPORT – EMBANKMENTS
CONSOLIDATION/STOP-GROUTING PLAN IN DIVERSION CHANNEL AREA (AS-BUILT)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

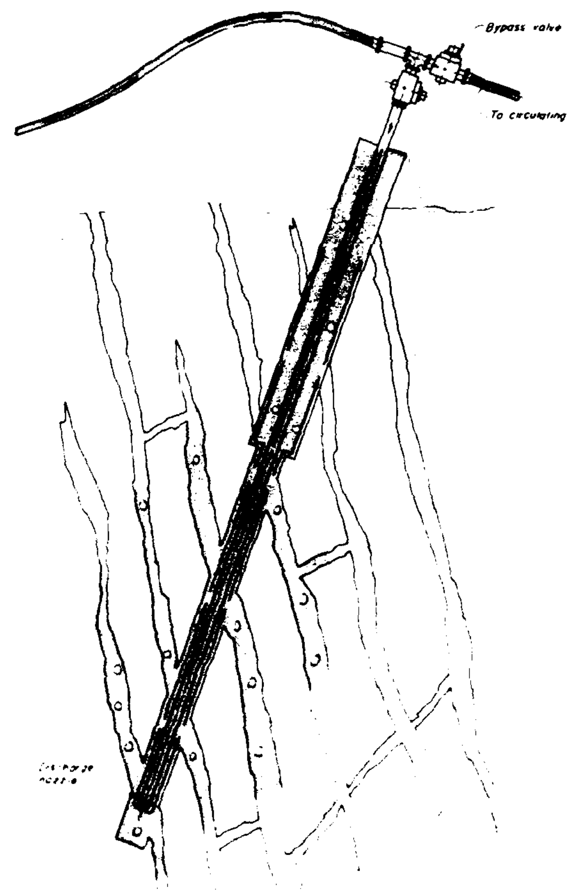
PLATE 109

REVISIONS



NOTE Boring is first drilled to depth (usually zone 1) using rotary and percussion methods. A mixture of air and water serves as a drilling medium.

1 DRILLING
NOT TO SCALE



NOTE Boring is next washed of cuttings and drill debris through an open ended pipe lowered to bottom of boring. A mixture of compressed air and water is used in the washing process.

2 WASHING
NOT TO SCALE

LEGEND

- Water and air
- Grout
- Rock

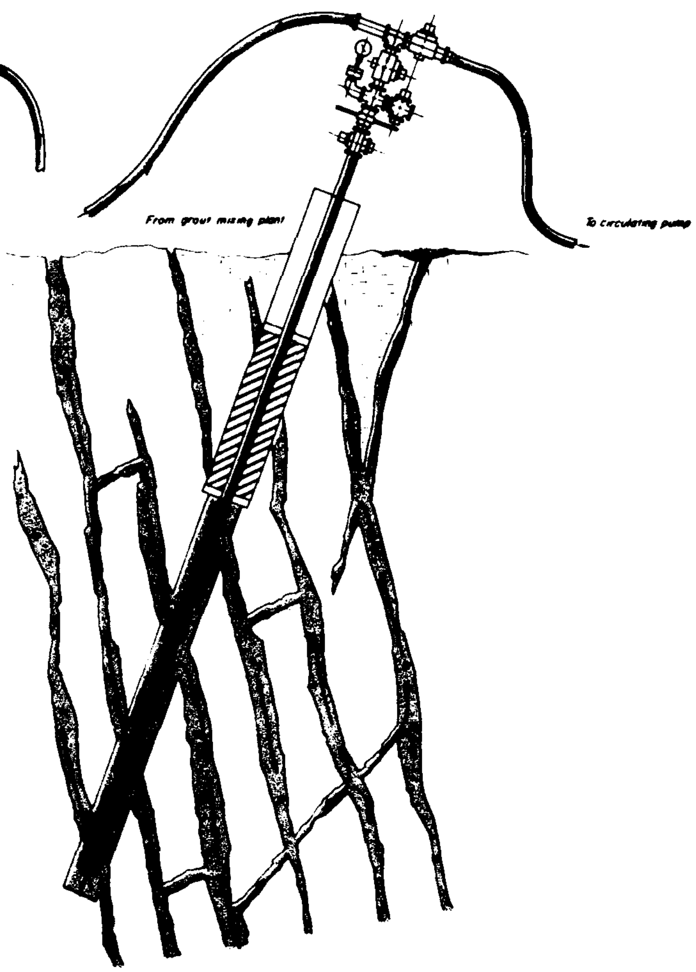
GENERAL NOTES

1. Fracture size, spacing and extent are grossly exaggerated for purpose of illustration.
2. Set 1 fractures are those having the largest occurrence throughout the project area. Refer to Plate 5.
3. Headers and washing tee shown may not have had these exact configurations in the field, however, the principle of bypass and recirculation was used.
4. Refer to Section 10.1.2 EE for description of water cement mix ratios used, refusal and other limiting criteria.
5. Communication between grout holes and surface leakage occurred randomly throughout the grouting process, records of which are maintained on field grouting reports.
6. 3" Black steel pipe was usually embedded and grouted into firm rock approximately 1 (one) foot. See Plate 12 for more detail.
7. This illustration is not intended to represent every field grouting event. A variety of circumstances encountered dictated the usage of other slightly different techniques.
8. For illustrative purpose, these drawings show idealized penetration of water and grout into rock joints.

debris through
A mixture
the washing process

NOTE After washing boring is pressure tested using water and a packer (mechanical or pneumatic). Water pressure is controlled at header as described in section 10-2-EE

3 WATER PRESSURE TESTING
NOT TO SCALE



NOTE Boring is then pressure grouted to refusal. Grout pressure is controlled at header as described in section 10-2-EE

4 GROUT INJECTION
NOT TO SCALE

SYMBOL	DATE	DESCRIPTION	APPR	BY
REVISIONS				

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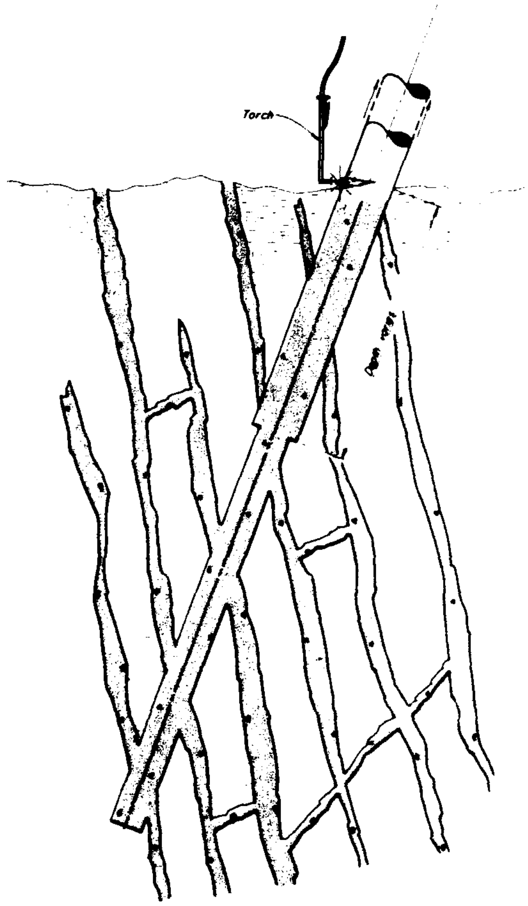
FOUNDATION REPORT
EMBANKMENTS
GROUTING PROCEDURES BY ZONE AND STAGE (SHEET 1 OF 2)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE	INVITATION NO.	FILE NO.	PLATE
F			110
SCALE NOT TO SCALE		SHEET	

110

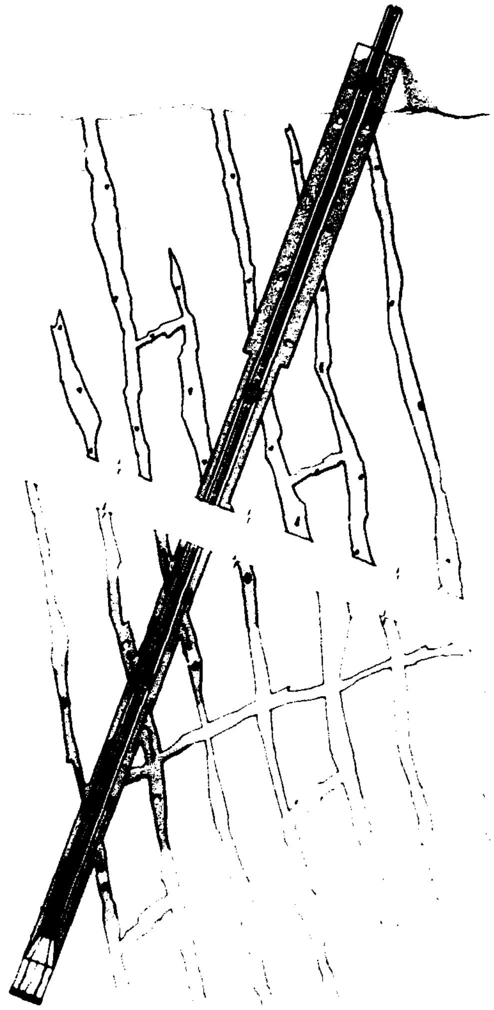
2



NOTE: After refusal is obtained, the grout placed is permitted time to stand and set properly. Should this zone be accepted, the 3" black steel pipe is either removed from the boring or burned off flush with the ground surface.

5 CURING

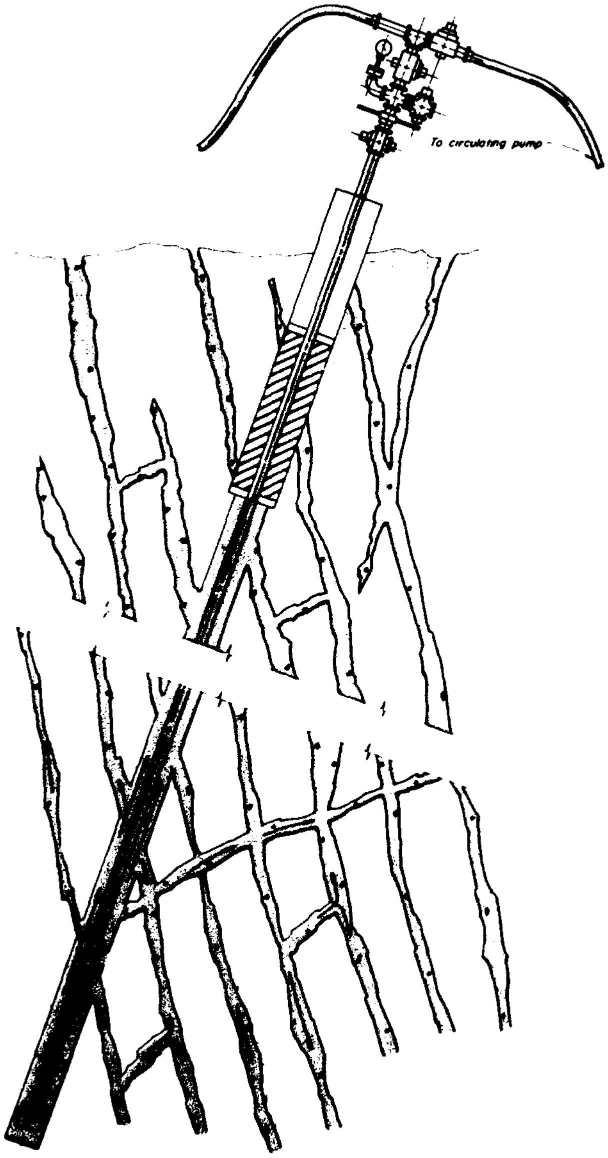
NOT TO SCALE



NOTE: Boring is redrilled and extended into deeper zones according to grout takes or prior designation as a set 1 or set 2 hole. (See contract curtain grouting profile Plate 8)

6 FLUSHING AND RE-DRILLING

NOT TO SCALE



NOTE All procedures are repeated as described until successive zones are accepted

7 REPEAT STEPS 2, 3, 4, 5 AND/OR 6

NOT TO SCALE

NOTES:

1. For notes see "General Notes" Plate 110

LEGEND

SYMBOL	ZONE	DESCRIPTION	DATE	BY

REVISIONS

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
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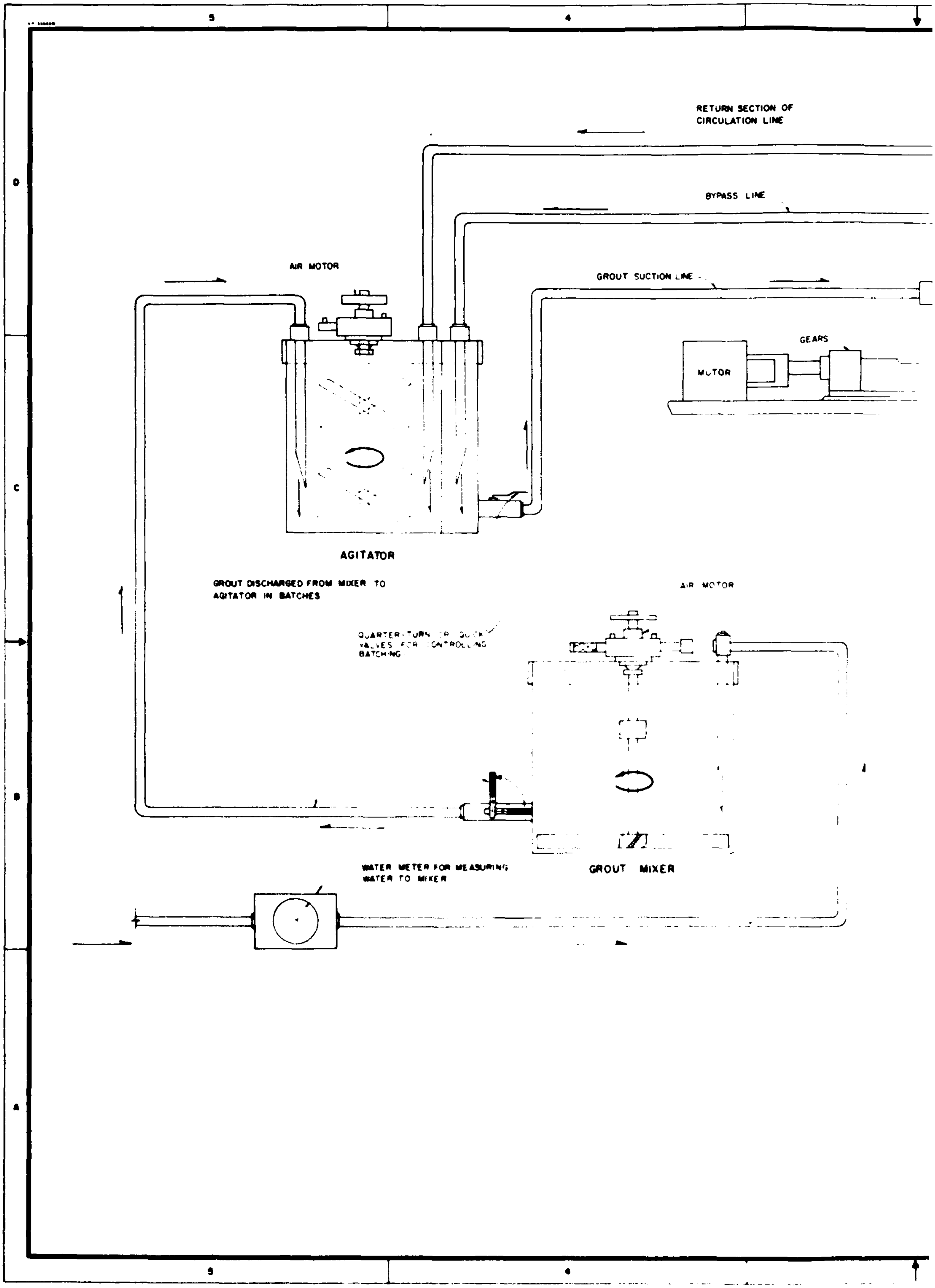
SAVANNAH, GEORGIA

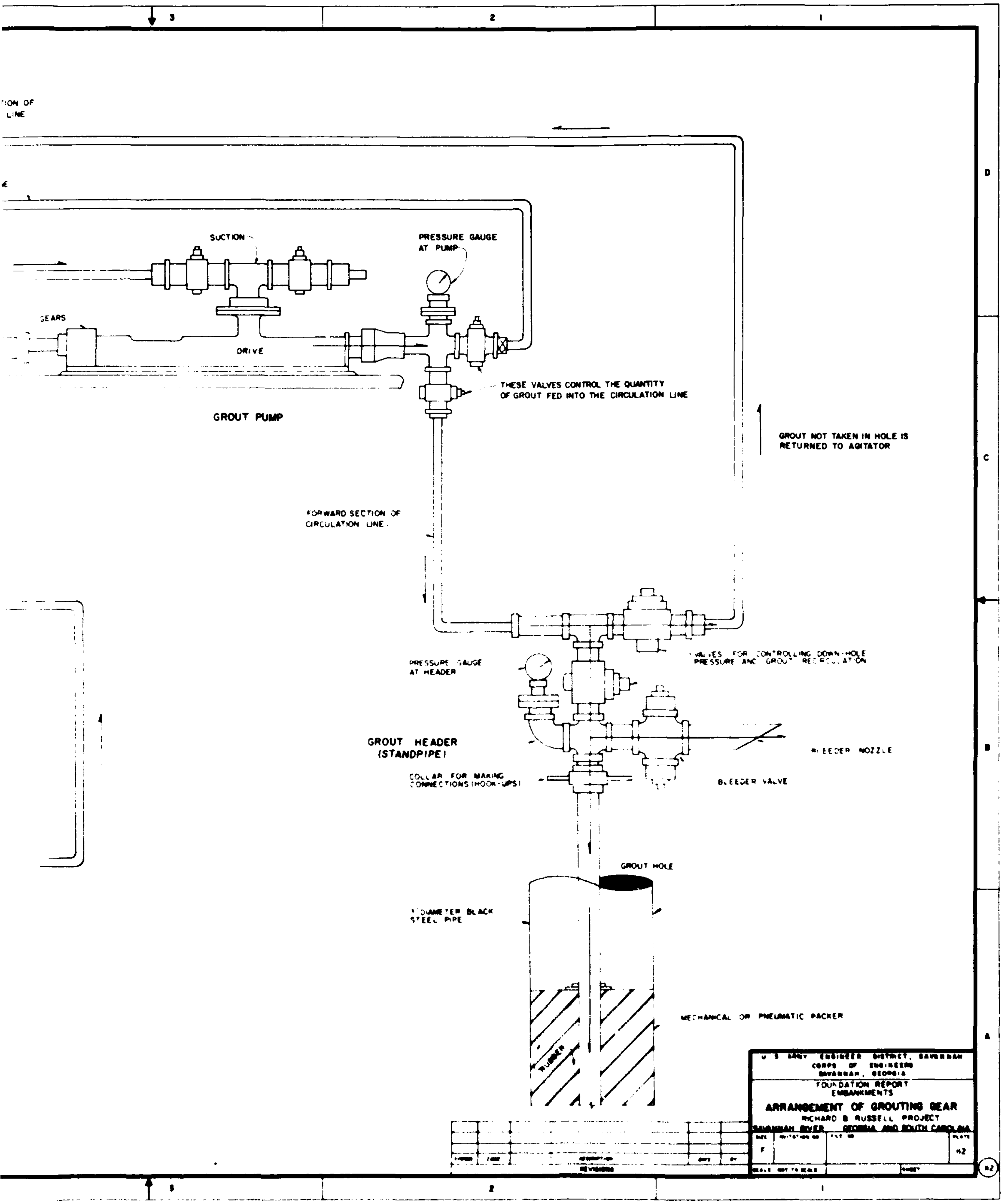
FOUNDATION REPORT
EMBANKMENTS
GROUTING PROCEDURES BY ZONE AND STAGE (SHEET 2 OF 2)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE	INVITATION NO.	FILE NO.	PLATE
F			111

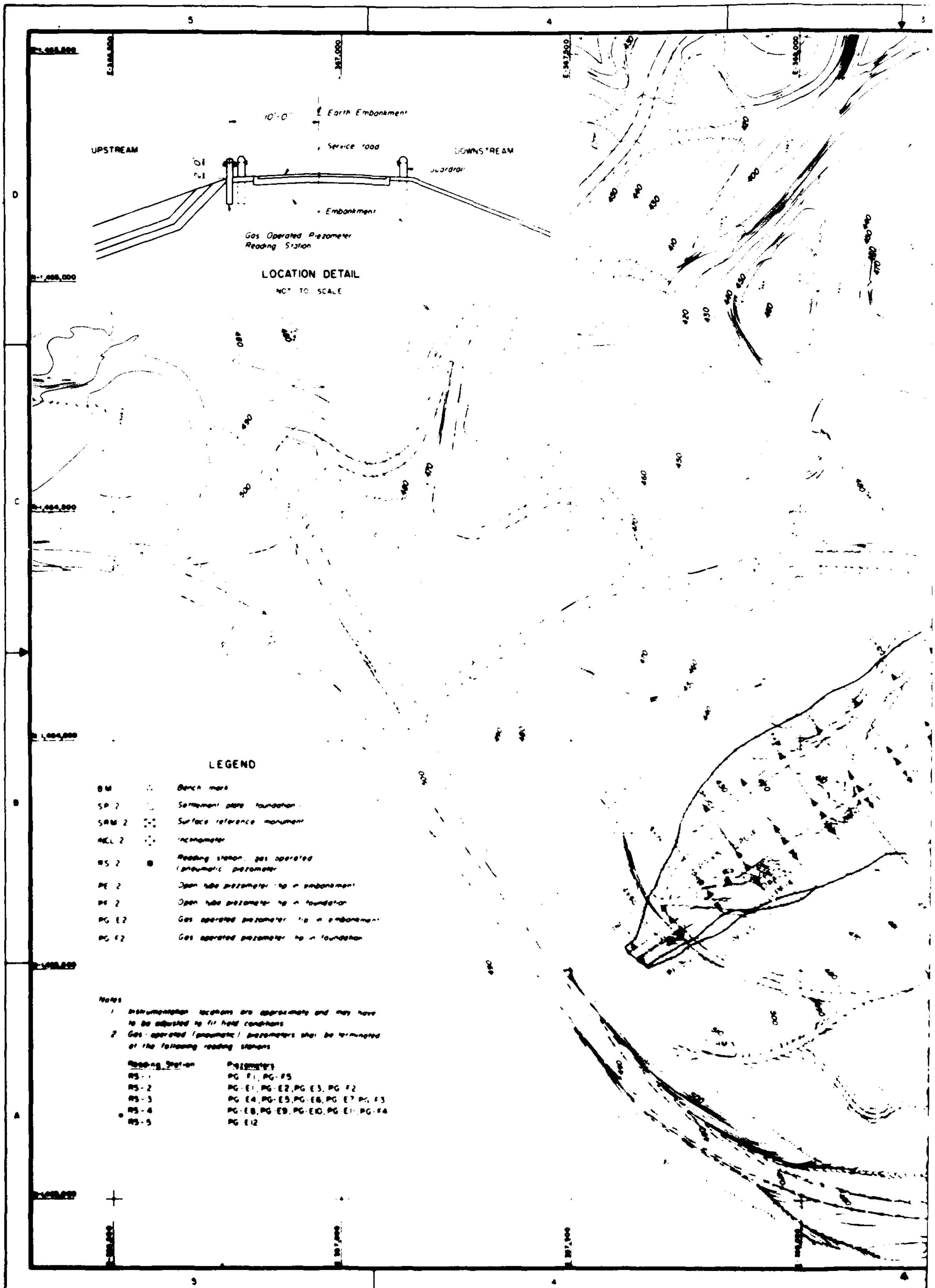
SCALE NOT TO SCALE — SHEET



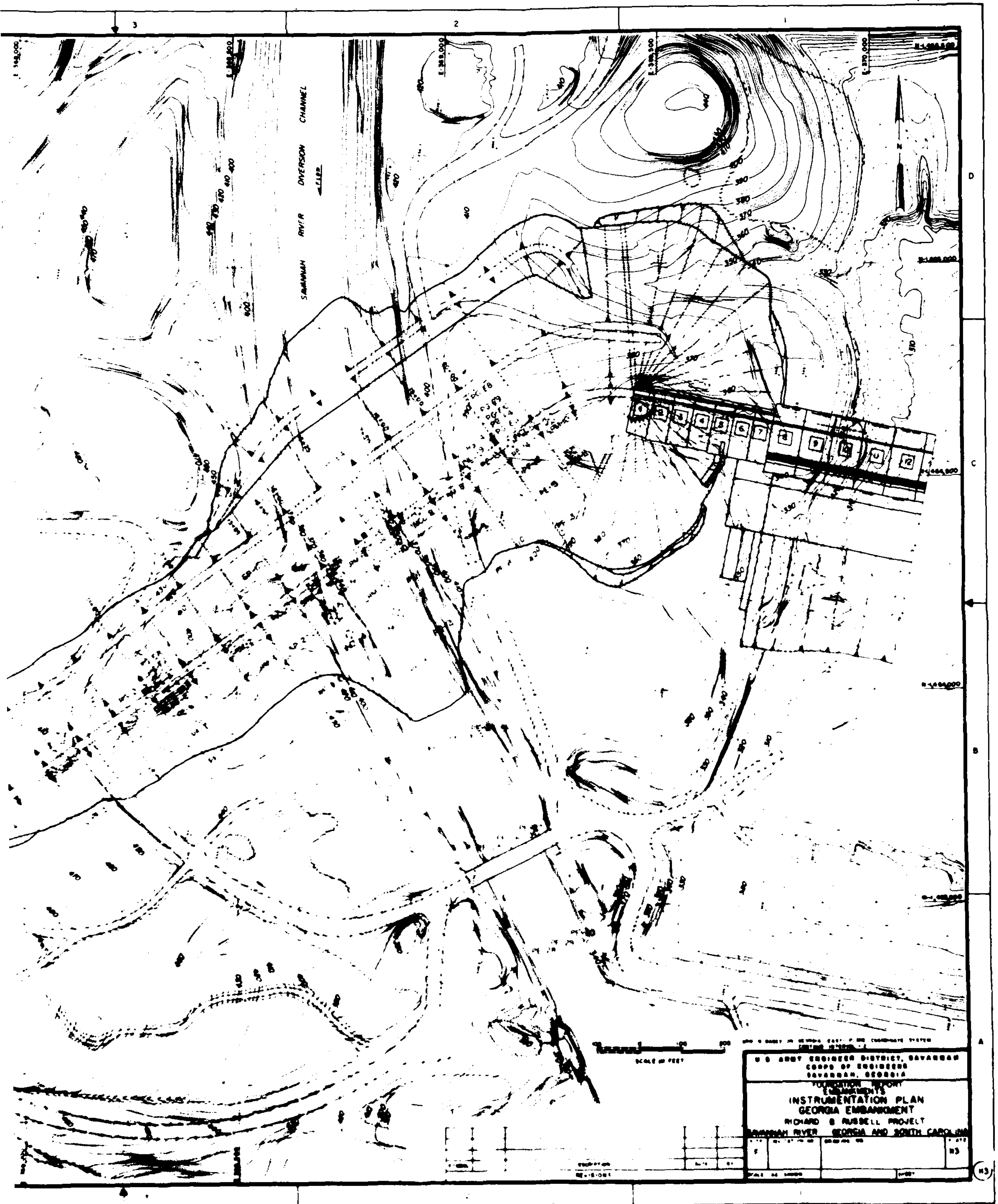
RETURN SECTION OF CIRCULATION LINE
BYPASS LINE
AIR MOTOR
GROUT SUCTION LINE
GEARS
MOTOR
AGITATOR
GROUT DISCHARGED FROM MIXER TO AGITATOR IN BATCHES
AIR MOTOR
QUARTER-TURN OR QUICK VALVES FOR CONTROLLING BATCHING
WATER METER FOR MEASURING WATER TO MIXER
GROUT MIXER

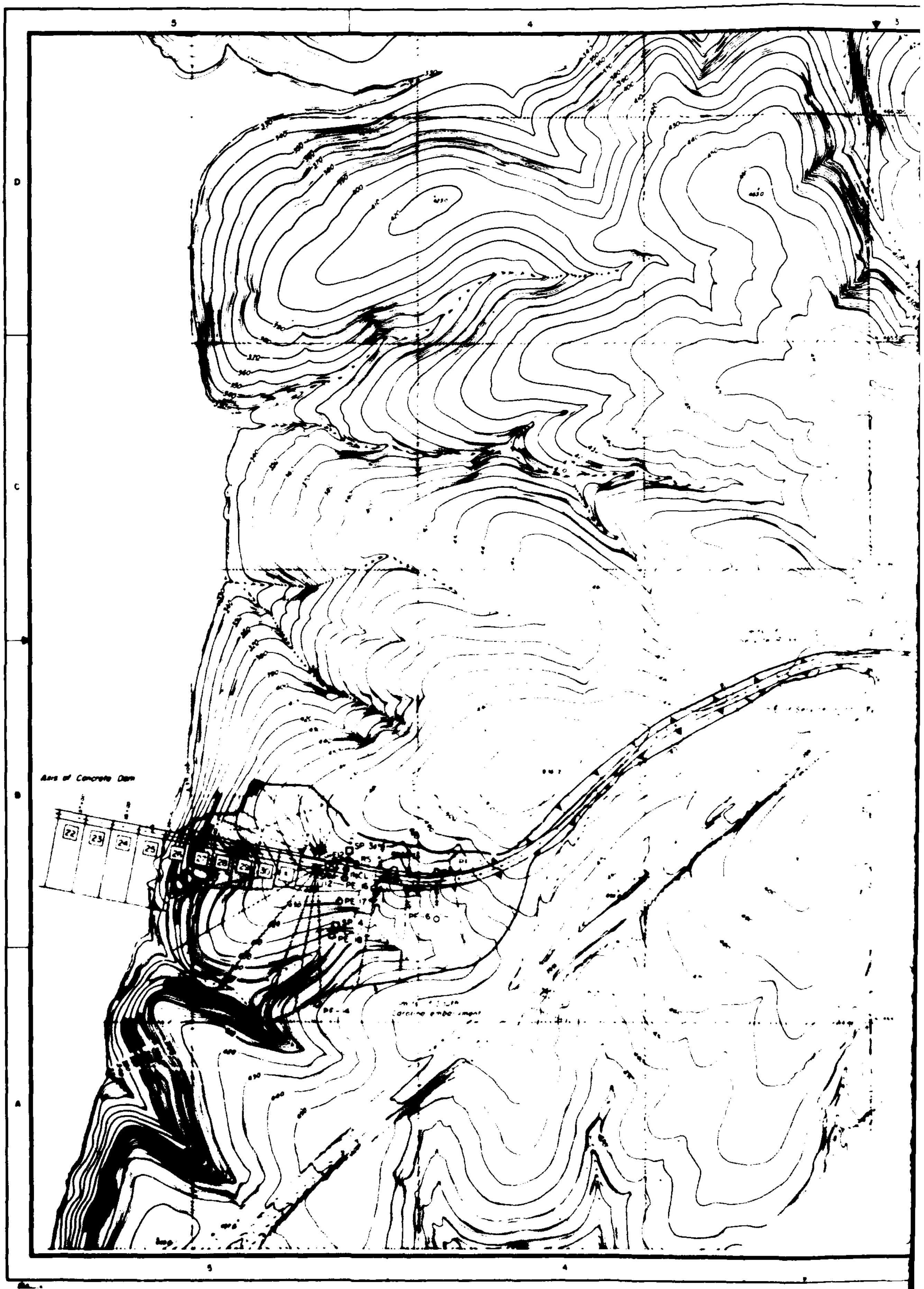


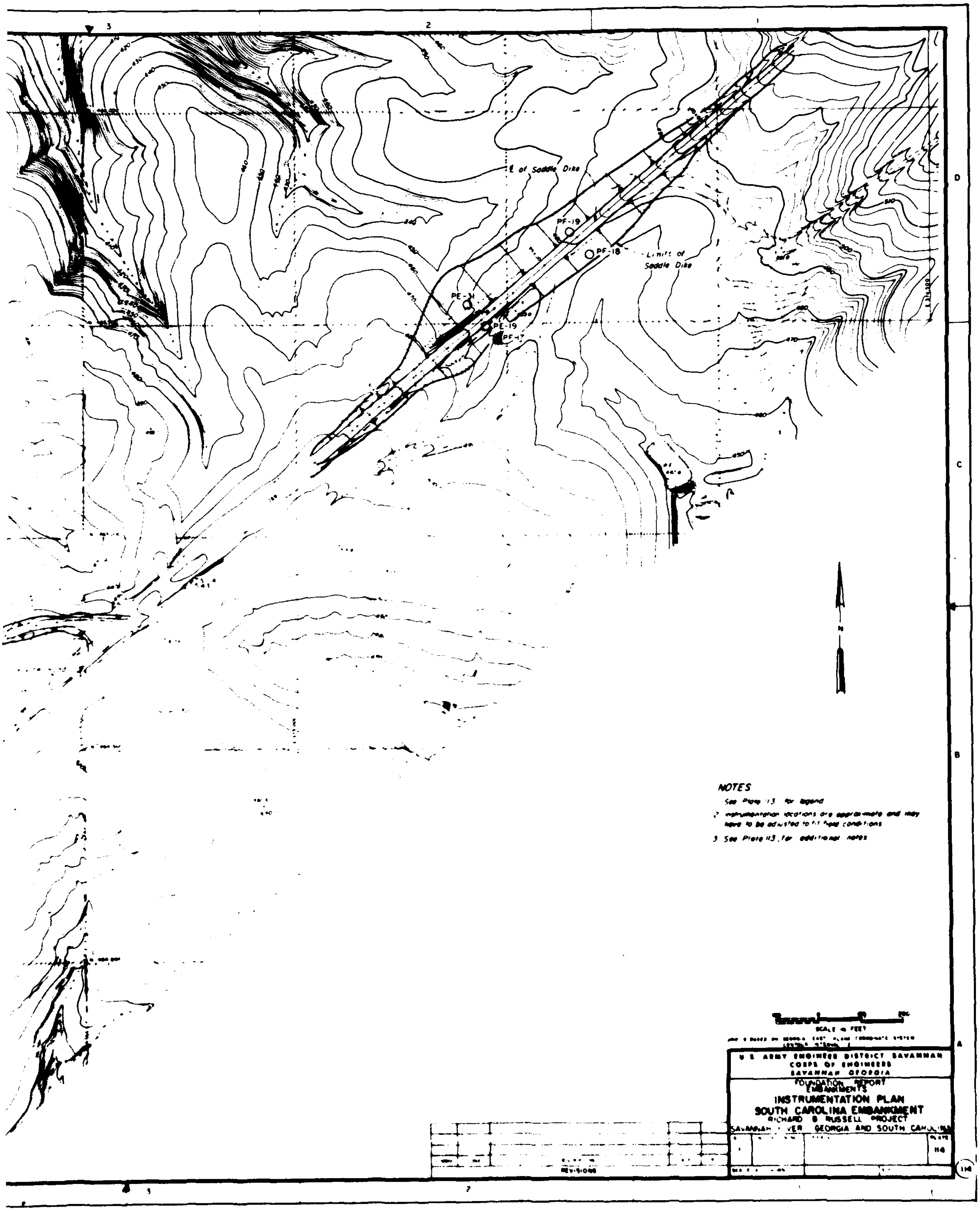
SUCTION
PRESSURE GAUGE AT PUMP
GEARS
DRIVE
THESE VALVES CONTROL THE QUANTITY OF GROUT FED INTO THE CIRCULATION LINE
GROUT PUMP
GROUT NOT TAKEN IN HOLE IS RETURNED TO AGITATOR
FORWARD SECTION OF CIRCULATION LINE
VALVES FOR CONTROLLING DOWN-HOLE PRESSURE AND GROUT RECIRCULATION
PRESSURE GAUGE AT HEADER
GROUT HEADER (STANDPIPE)
BLEEDER NOZZLE
COLLAR FOR MAKING CONNECTIONS (HOOK-UPS)
BLEEDER VALVE
GROUT HOLE
DIAMETER BLACK STEEL PIPE
MECHANICAL OR PNEUMATIC PACKER
RUBBER
REVISIONS
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
ARRANGEMENT OF GROUTING GEAR
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
F
H2
SCALE NOT TO SCALE
H2



10'-0"
℄ Earth Embankment
UPSTREAM
Service road
DOWNSTREAM
Embankment
Gas Operated Piezometer
Reading Station
LOCATION DETAIL
NOT TO SCALE
LEGEND
BM Bench mark
SP 2 Settlement plate (foundation)
SRM 2 Surface reference monument
INCL 2 Inclinometer
RS 2 Reading station, gas operated (pneumatic) piezometer
PE 2 Open tube piezometer tip in embankment
PF 2 Open tube piezometer tip in foundation
PG E2 Gas operated piezometer tip in embankment
PG F2 Gas operated piezometer tip in foundation
Notes
1 Instrumentation locations are approximate and may have to be adjusted to fit field conditions
2 Gas operated (pneumatic) piezometers shall be terminated at the following reading stations
Reading Station
Piezometers
RS-1
PG-F1, PG-F5
RS-2
PG-E1, PG-E2, PG-E3, PG-F2
RS-3
PG-E4, PG-E5, PG-E6, PG-E7, PG-F3
RS-4
PG-E8, PG-E9, PG-E10, PG-E11, PG-F4
RS-5
PG-E12

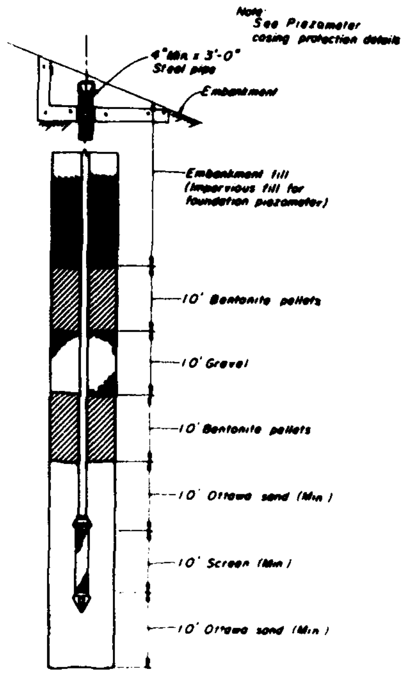

SAVANNAH RIVER DIVERSION CHANNEL
SCALE IN FEET
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
EMBANKMENTS
INSTRUMENTATION PLAN
GEORGIA EMBANKMENT
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
H3


Axis of Concrete Dam
22
23
24
25



℄ of Saddle Dike
PF-19
PF-18
Limits of Saddle Dike
PE-31
PE-19
PF-17
NOTES
See Plate 113 for legend
2 Instrumentation locations are approximate and may have to be adjusted to fit field conditions
3 See Plate 113 for additional notes
SCALE IN FEET
U.S. ARMY ENGINEER DISTRICT SAVANNAH
CORPS OF ENGINEERS
SAVANNAH GEORGIA
FOUNDATION REPORT
EMBANKMENTS
INSTRUMENTATION PLAN
SOUTH CAROLINA EMBANKMENT
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
PLATE
114
REVISIONS
114

2

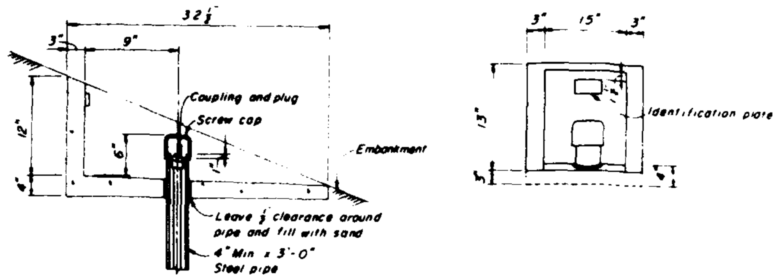


EMBANKMENT SLOPE

INSTALLATION OPEN-SYSTEM PIEZOMETER

NOT TO SCALE

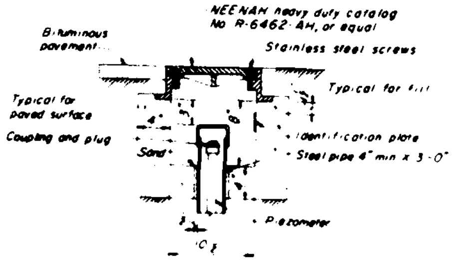
Note:
Piezometer and inclinometer/settlement gage are government furnished details are for information only



SECTION THRU LONG AXIS

FRONT ELEVATION

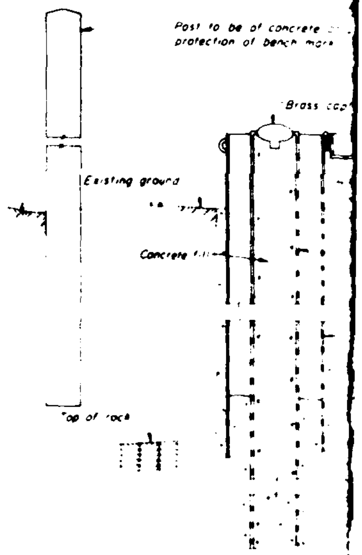
FOR EMBANKMENT SLOPE



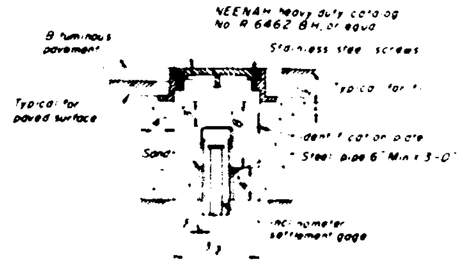
FOR TOP OF FILL OR PAVEMENT

CASING PROTECTION DETAILS FOR PIEZOMETERS

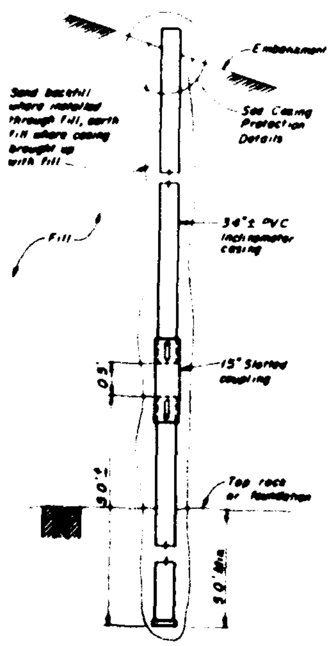
NOT TO SCALE



BENCHMARK DETA...

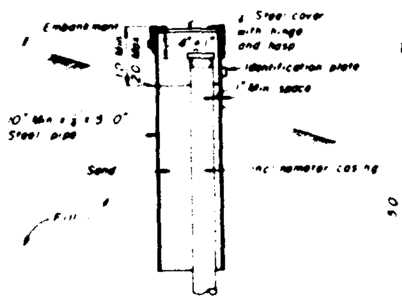


FOR TOP OF FILL OR PAVEMENT



INCLINOMETER/SETTLEMENT GAGE INSTALLATION

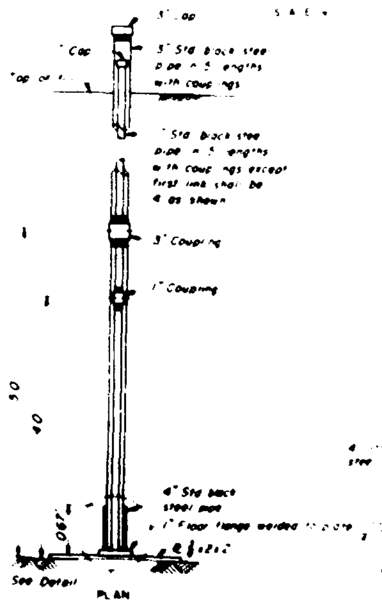
SCALE 1"=1.0"



FOR EMBANKMENT SLOPE

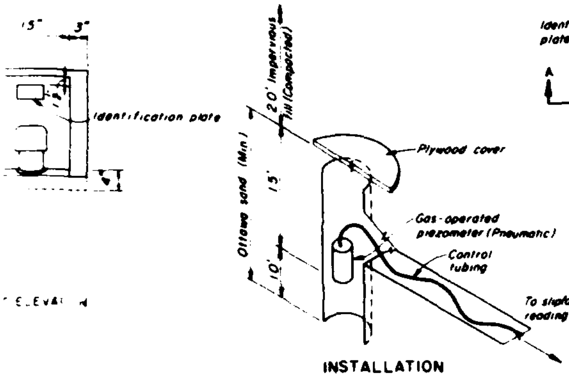
CASING PROTECTION DETAILS FOR INCLINOMETER/SETTLEMENT GAGE

NOT TO SCALE



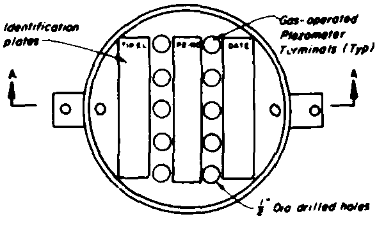
SETTLEMENT PLATE

SCALE ... 0"

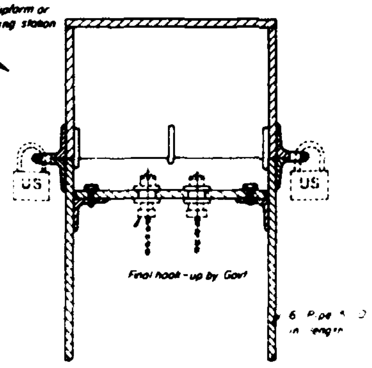


INSTALLATION
GAS-OPERATED PIEZOMETER

NOT TO SCALE



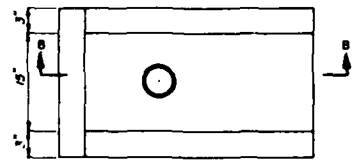
PLAN



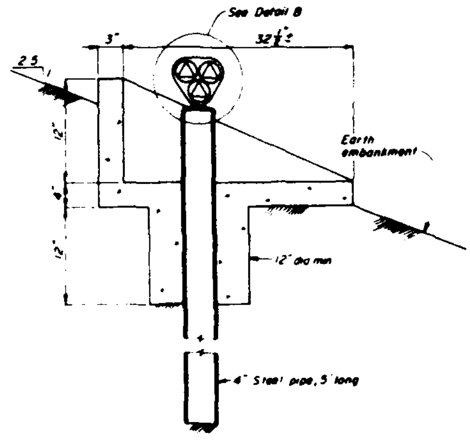
A-A

DETAIL A

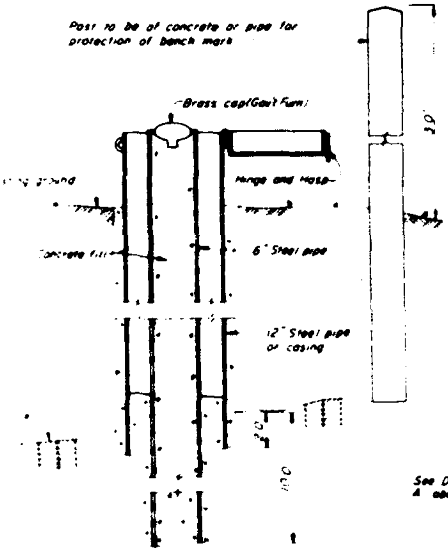
NOT TO SCALE



PLAN

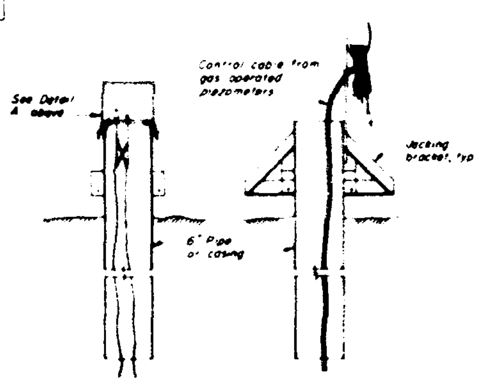


B-B



BENCHMARK DETAIL

SCALE

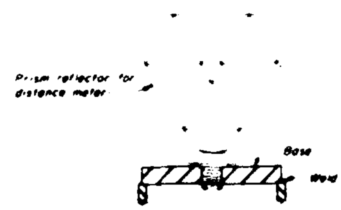


CONVERTED TO PERMANENT READING STATION

FOR EXTENDING CABLES DURING CONSTRUCTION

GAS-OPERATED PIEZOMETER CABLE SLIPFORM

NOT TO SCALE



NOT TO SCALE

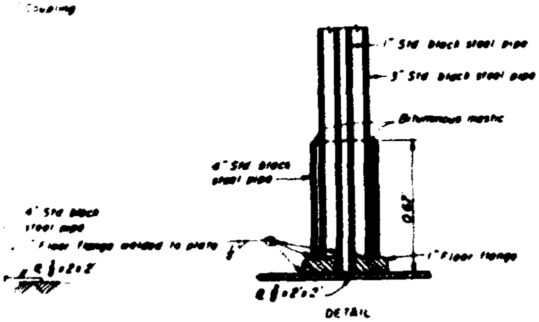
DETAIL B

NOTE

Contractor shall submit for approval plan and details of proposed installation procedures for the Surface Reference Monuments. Equipment for monitoring movements shall be Hewlett-Packard distance meter Model 3820 A, or equal. The Contractor shall obtain from the equipment manufacturer and furnish sufficient "Model Numbers, Data and Equipment Specifications" for review, and the character and efficiency of this equipment shall be subject to the approval of the Contracting Officer. Regardless of the equipment used for readings during construction, base shall be threaded to accept Hewlett-Packard prism adapter Model H412E

SURFACE REFERENCE MONUMENT

SCALE 1 1/2" = 1'-0"



DETAIL

SETTLEMENT PLATE

SCALE

U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
EMBANKMENTS

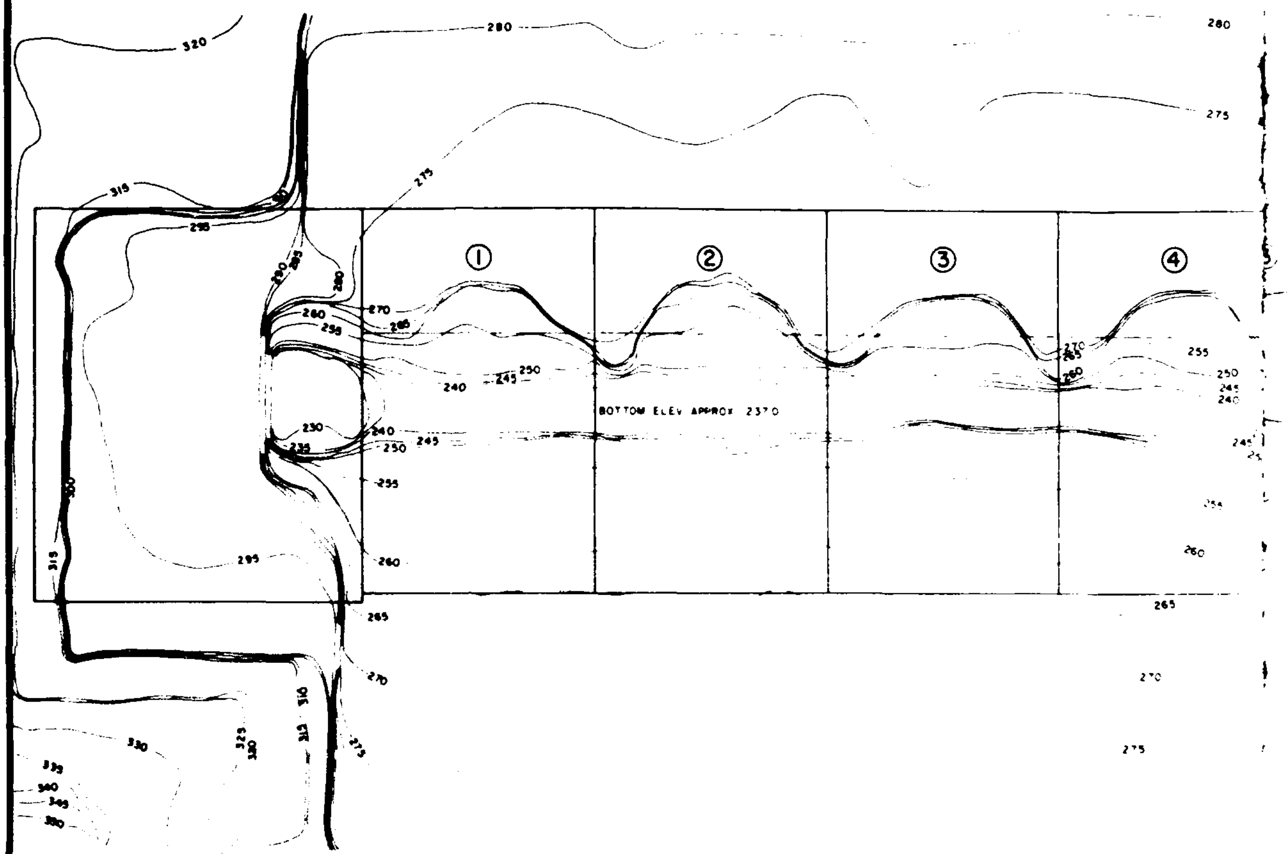
INSTRUMENTATION DETAILS
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER — GEORGIA AND SOUTH CAROLINA

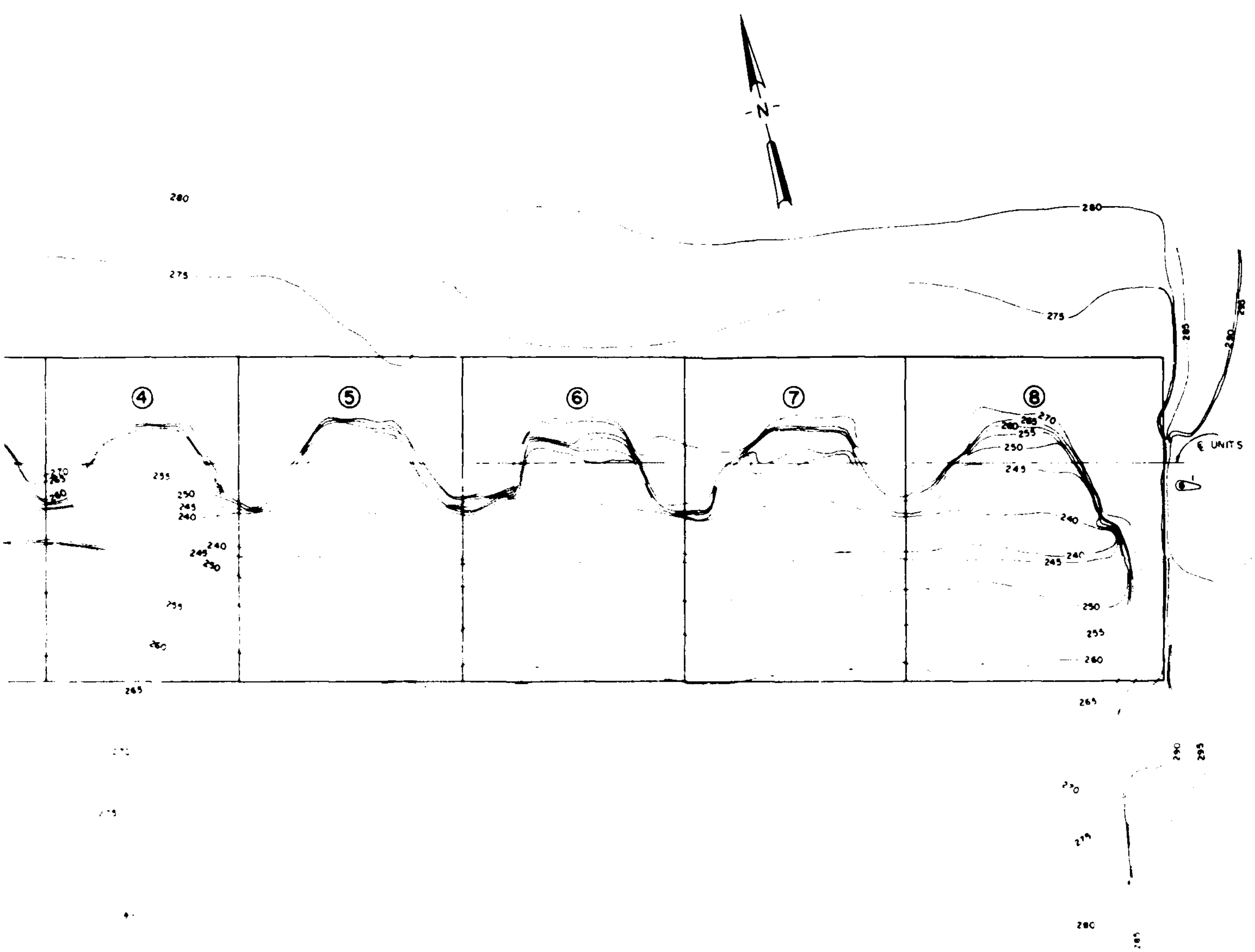
PLATE 115

SCALE AS SHOWN

REVISIONS

CORPS OF ENGINEERS





NOTE: THE CONTOURS SHOWN ARE BASED ON DATA OBTAINED FOLLOWING COMPLETION OF THE EXCAVATION BY A PREVIOUS CONTRACTOR [illegible]. THEY ARE APPROXIMATE AND MAY NOT REPRESENT THE EXACT CONFIGURATION. THESE DATA ARE PRESENTED FOR INFORMATIONAL PURPOSES.

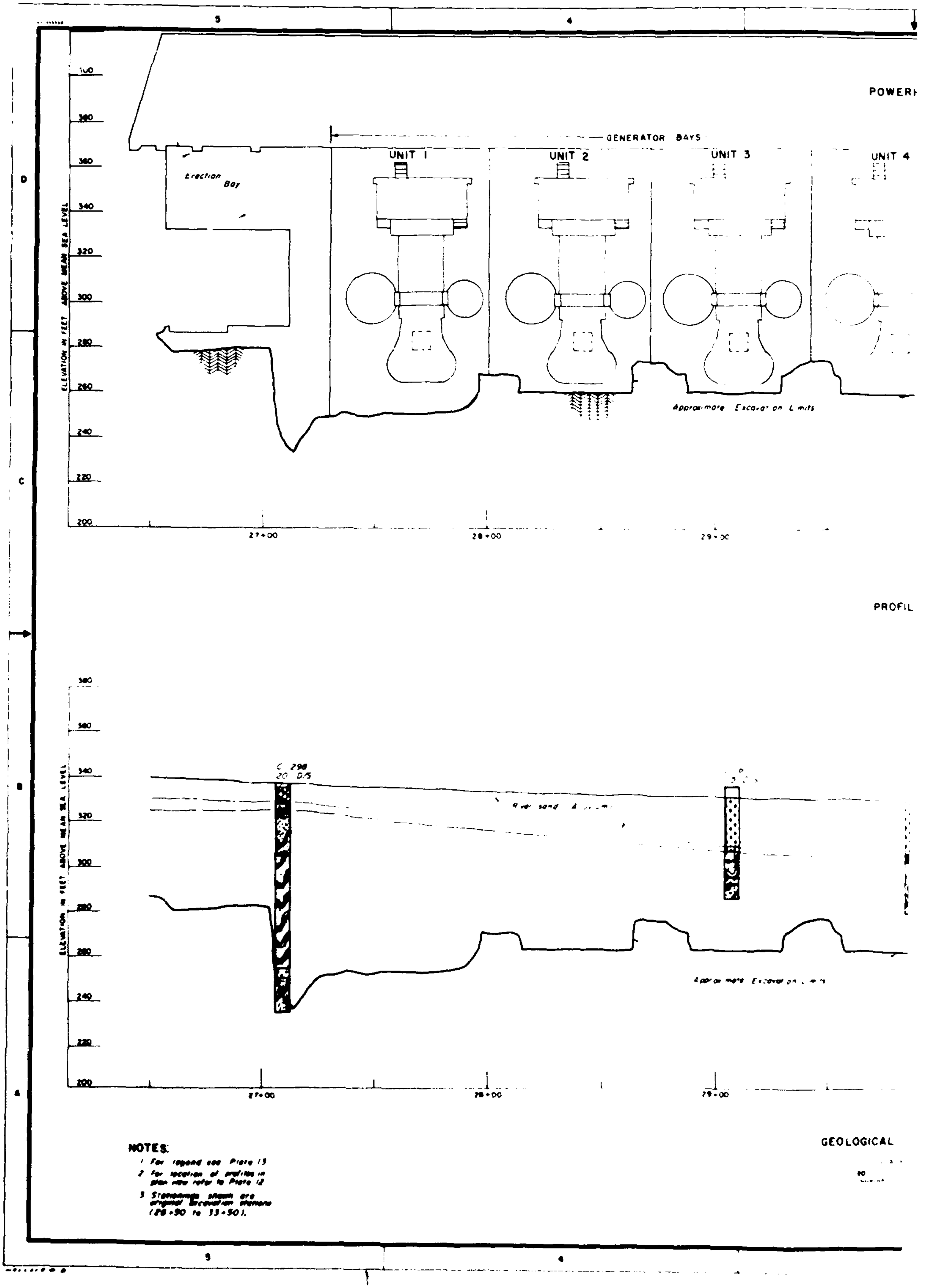
FOUNDATION REPORT
POWERHOUSE
POWERHOUSE
TOPOGRAPHY
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

F

116

20

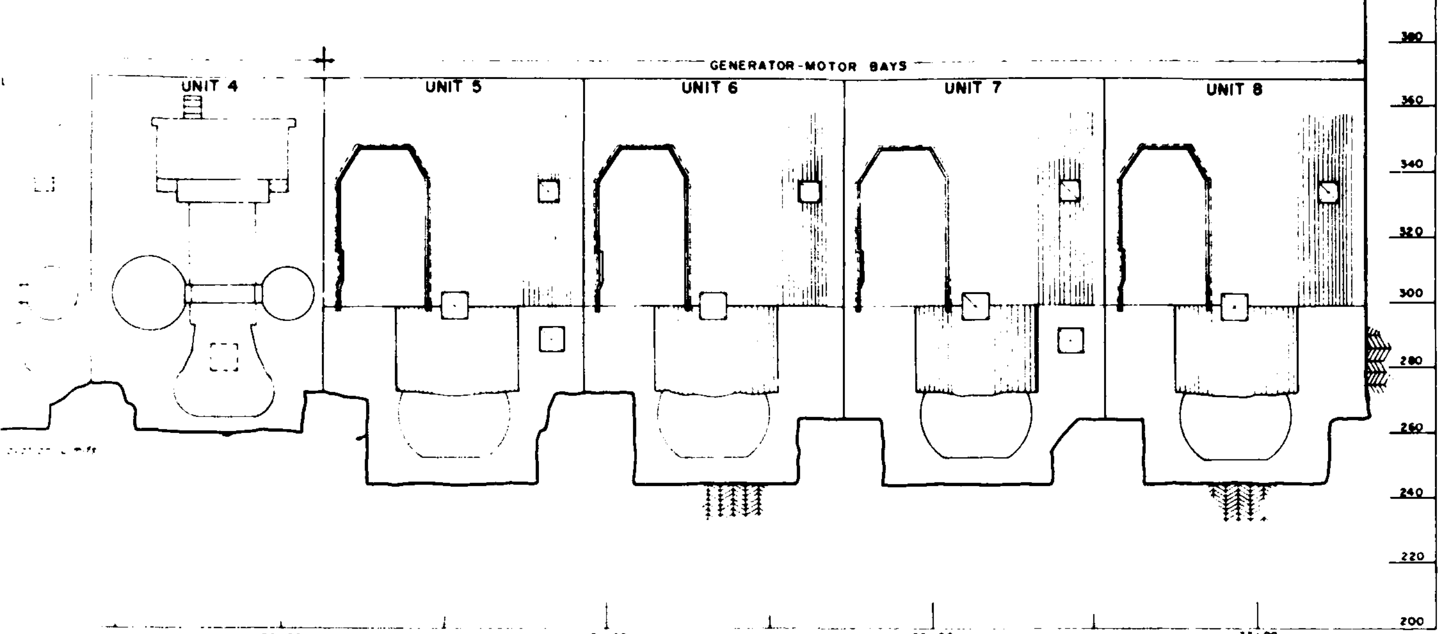
REVISIONS



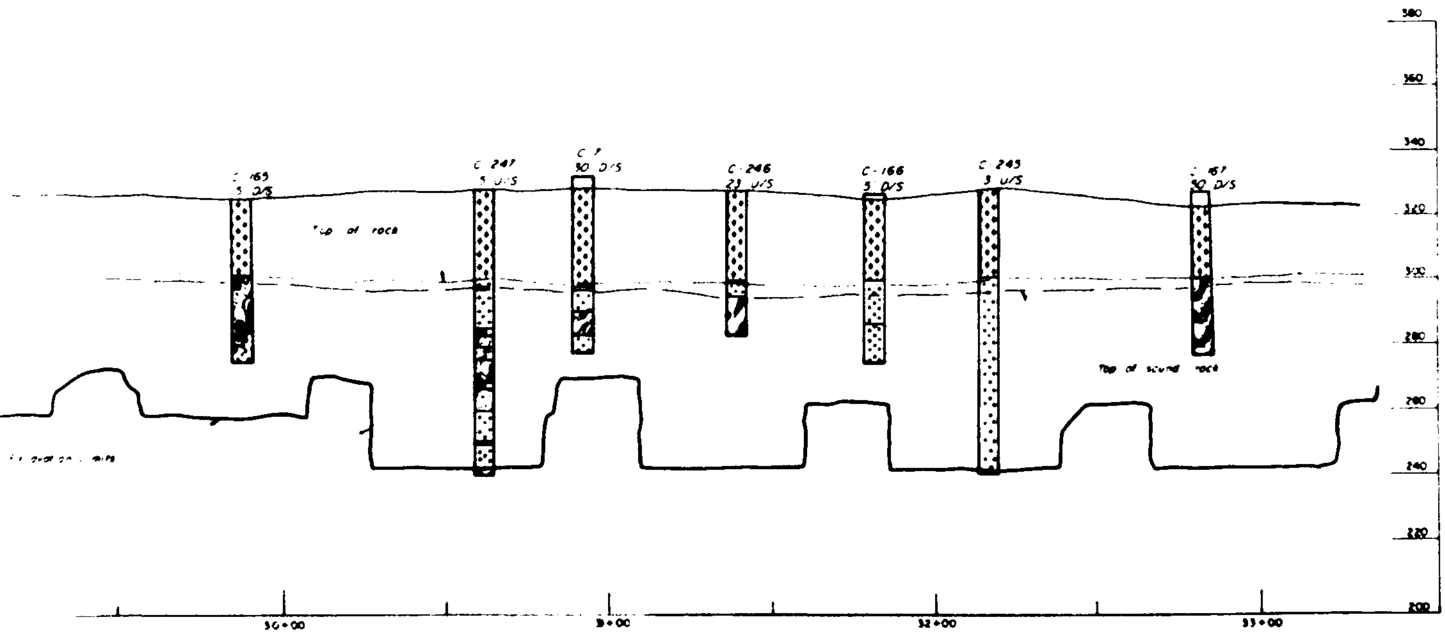
NOTES:

1. For legend see Plate 13
2. For location of profiles in plan view refer to Plate 12
3. Stationings shown are original excavation stations (28+50 to 33+50).

POWERHOUSE



PROFILE C-C'



GEOLOGICAL PROFILE C-C'

SCALE 1"=20'

U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
POWERHOUSE

GEOLOGICAL PROFILE C-C'

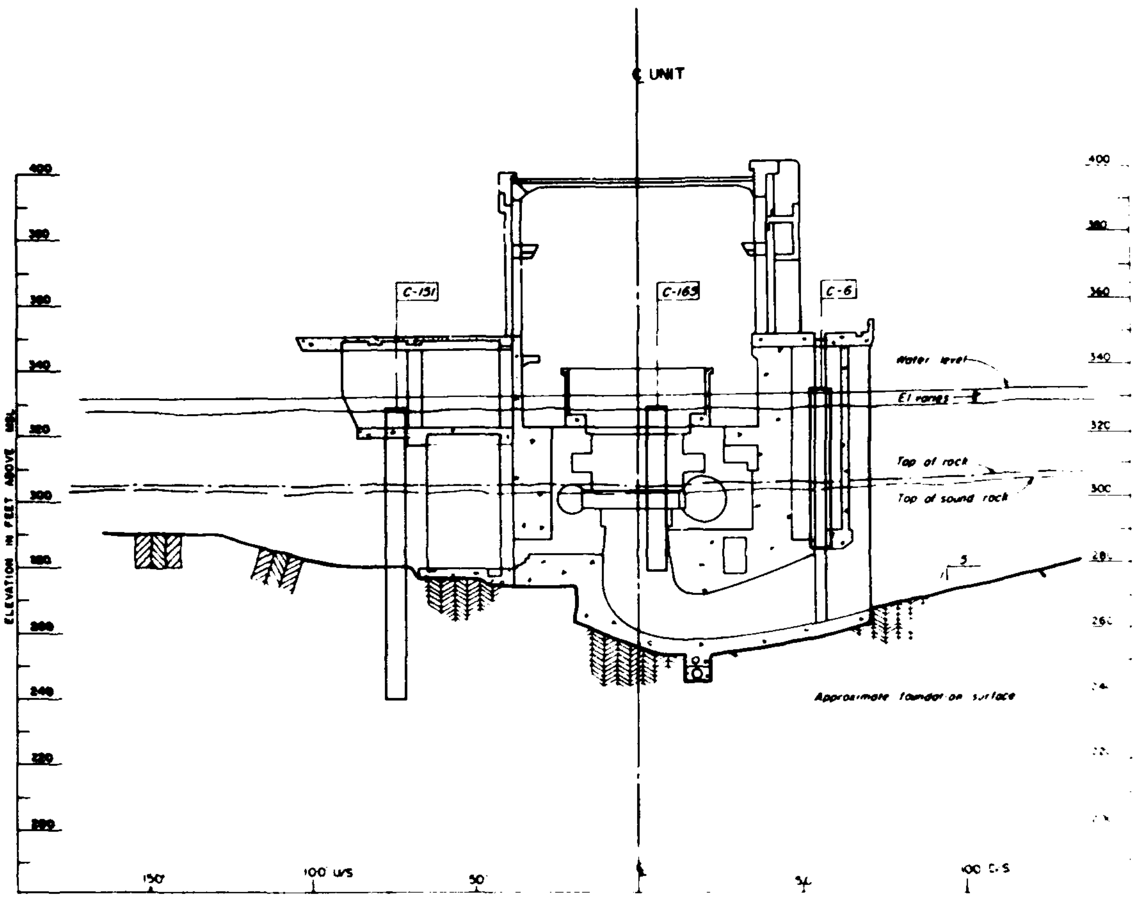
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

SIZE			PLATE
F			117

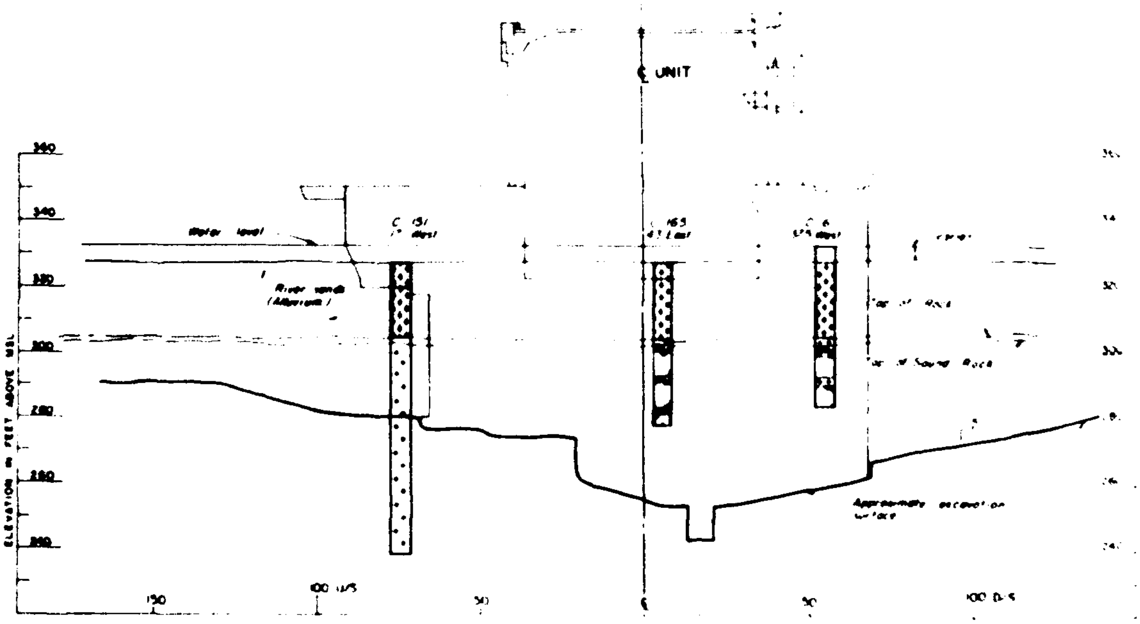
SCALE 1"=20'

REVISIONS

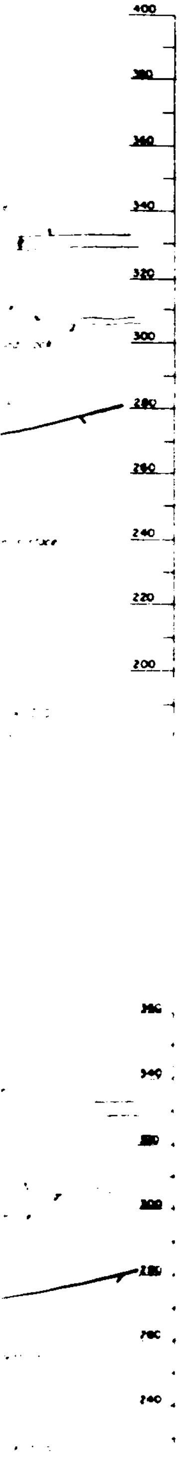
117

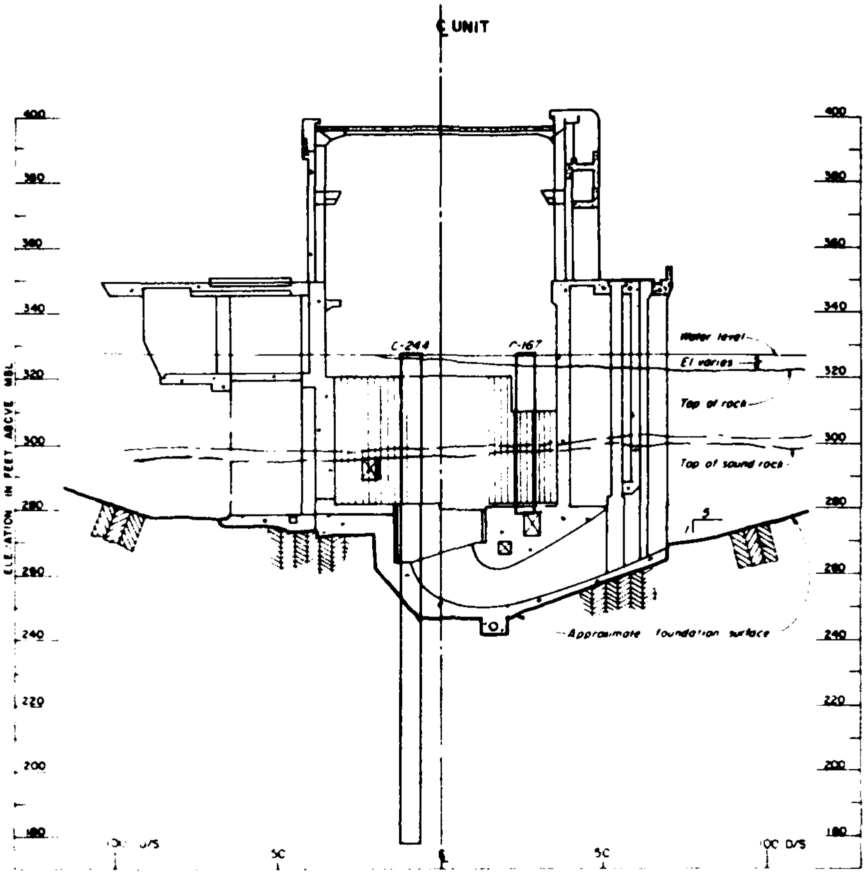


CROSS-SECTION D-D'
(TYPICAL GENERATOR UNIT)

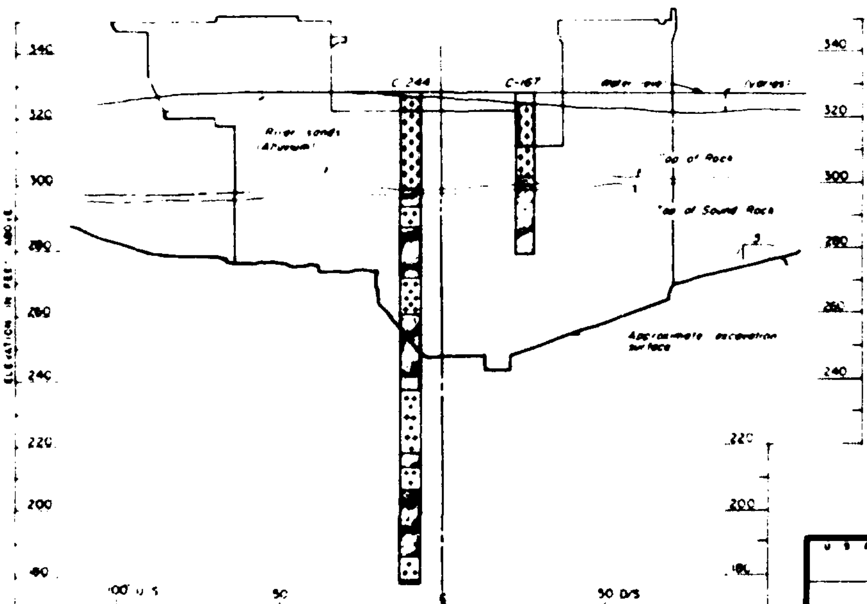


GEOLOGICAL SECTION D-D'





CROSS-SECTION E-E'
(TYPICAL GENERATOR-MOTOR UNIT)



GEOLOGICAL SECTION E-E'

NOTES

1. For location of sections in plan view, refer to Plate 12
2. For legend see Plate 13
3. Geological logs of borings are maintained in the files of SASEN-GG

SCALE 1" = 20'

U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

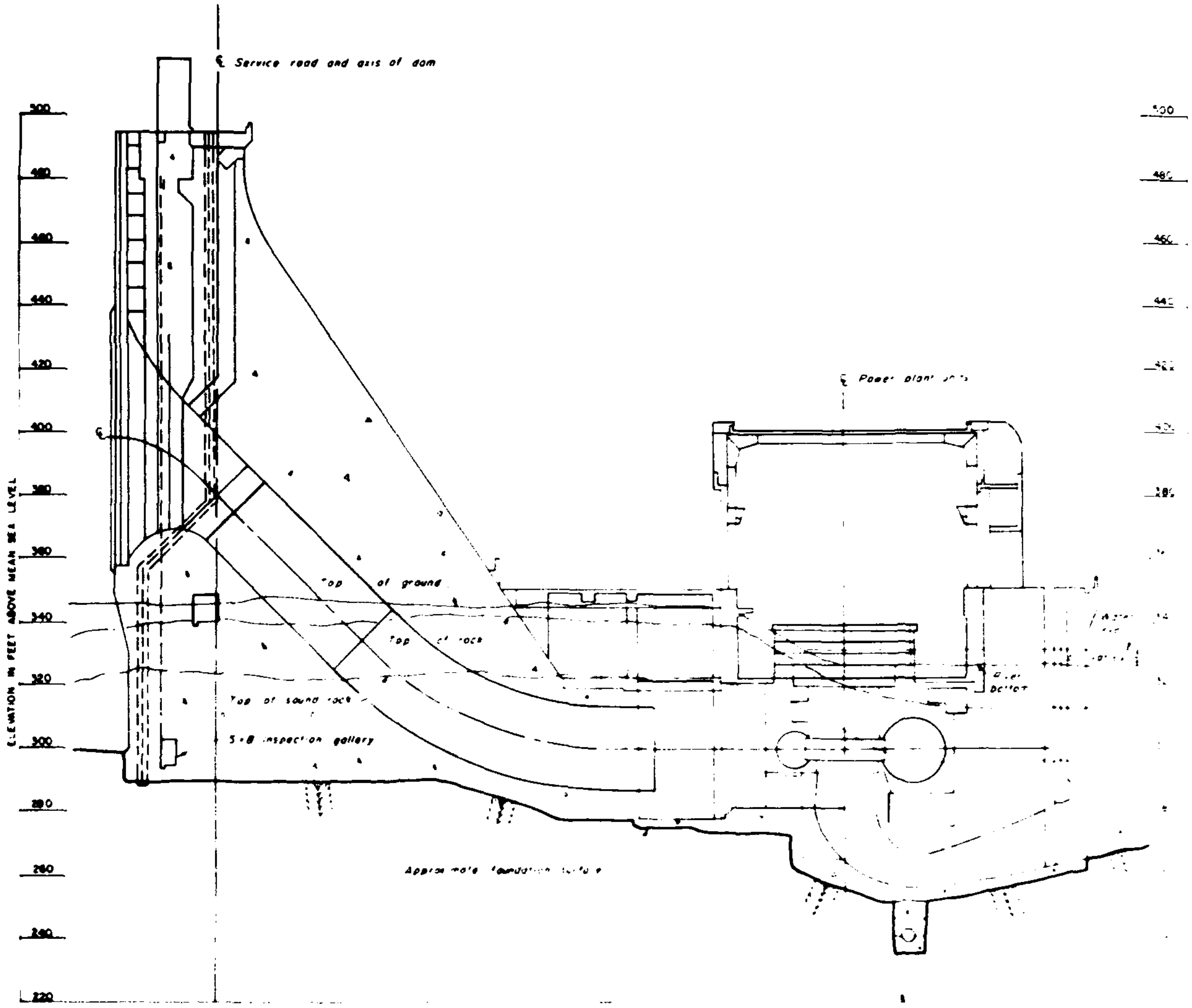
FOUNDATION REPORT
POWERHOUSE

GEOLOGICAL CROSS-SECTIONS

RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER — GEORGIA AND SOUTH CAROLINA

PLATE 11B

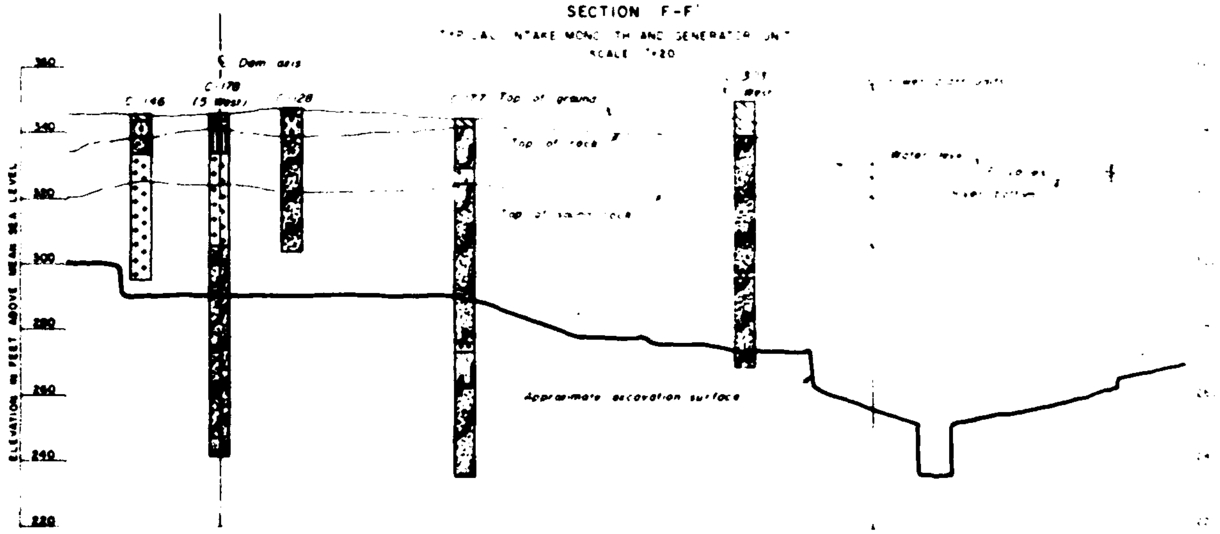
REVISIONS



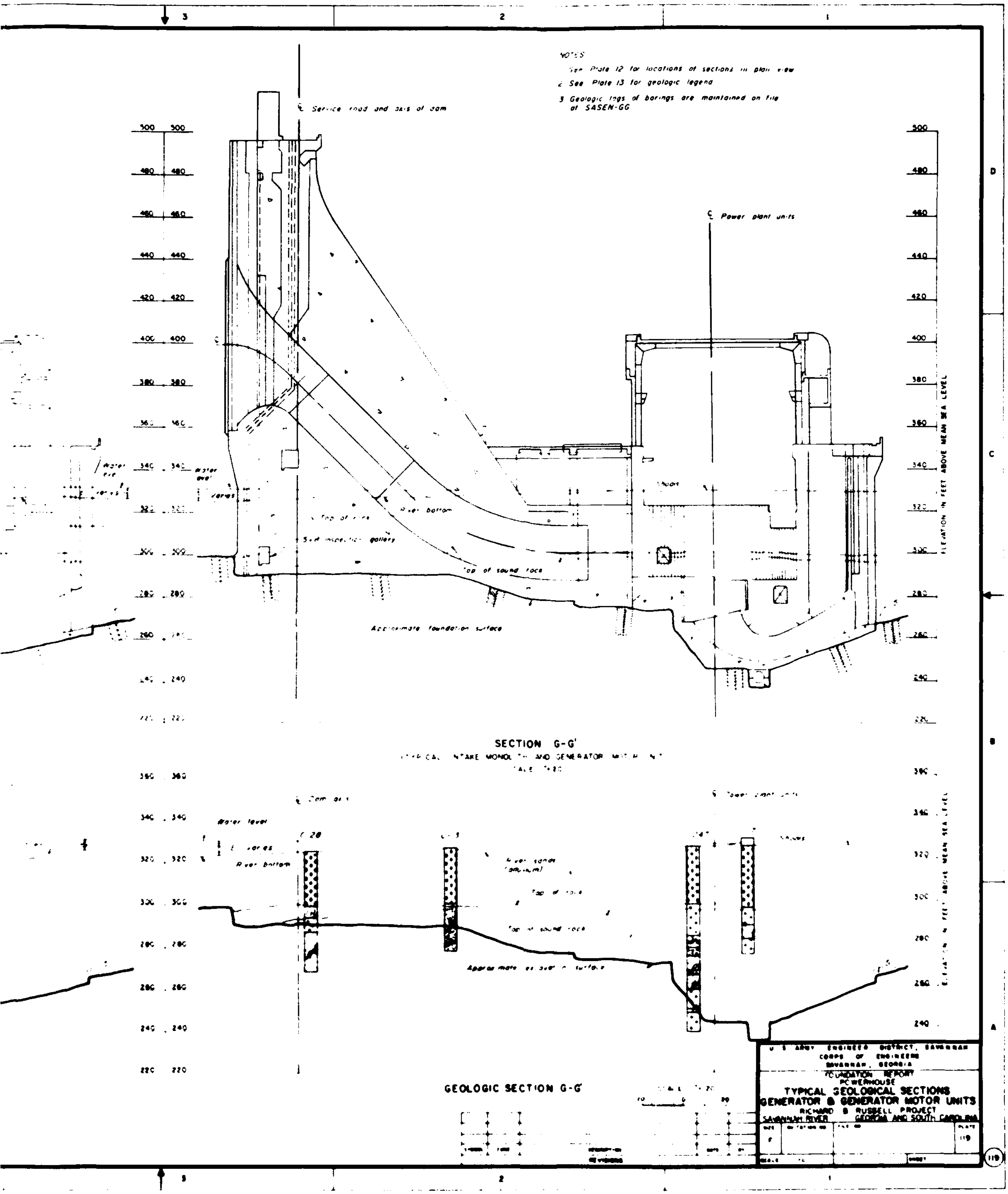
SECTION F-F'

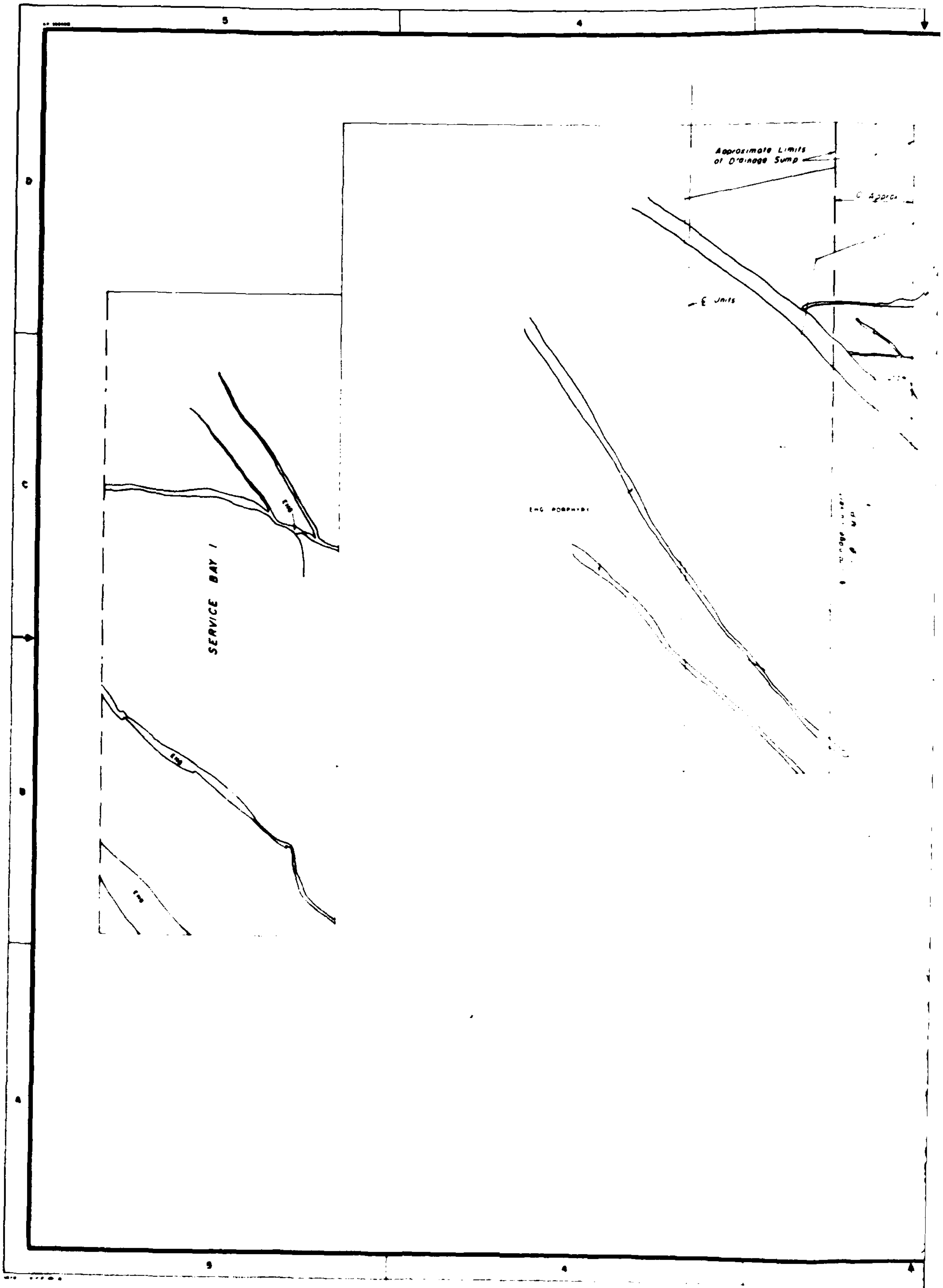
TYPICAL INTAKE MONOLITH AND GENERATOR UNIT

SCALE 1"=20'

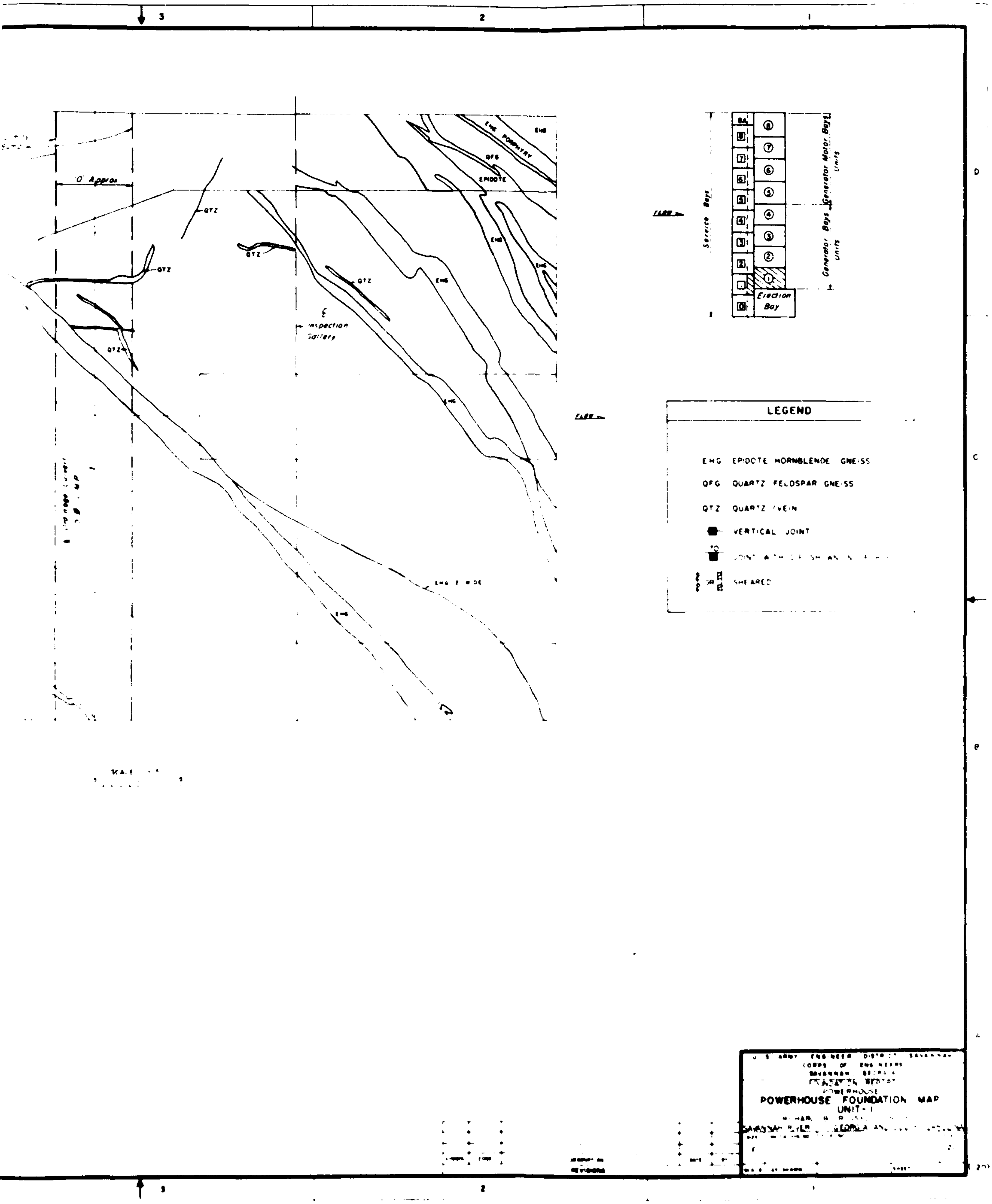


GEOLOGIC SECTION F-F'


NOTES
1 See Plate 12 for locations of sections in plan view
2 See Plate 13 for geologic legend
3 Geologic logs of borings are maintained on file at SASEN-GG
℄ Service road and axis of dam
℄ Power plant units
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
Water level
River bottom
Top of rock
5x8 inspection gallery
Top of sound rock
Approximate foundation surface
SECTION G-G'
℄ Dam axis
River sands (alluvium)
Top of rock
Top of sound rock
Approximate excavation surface
GEOLOGIC SECTION G-G'
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
FOUNDATION REPORT
POWERHOUSE
TYPICAL GEOLOGICAL SECTIONS
GENERATOR & GENERATOR MOTOR UNITS
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
REVISIONS
119

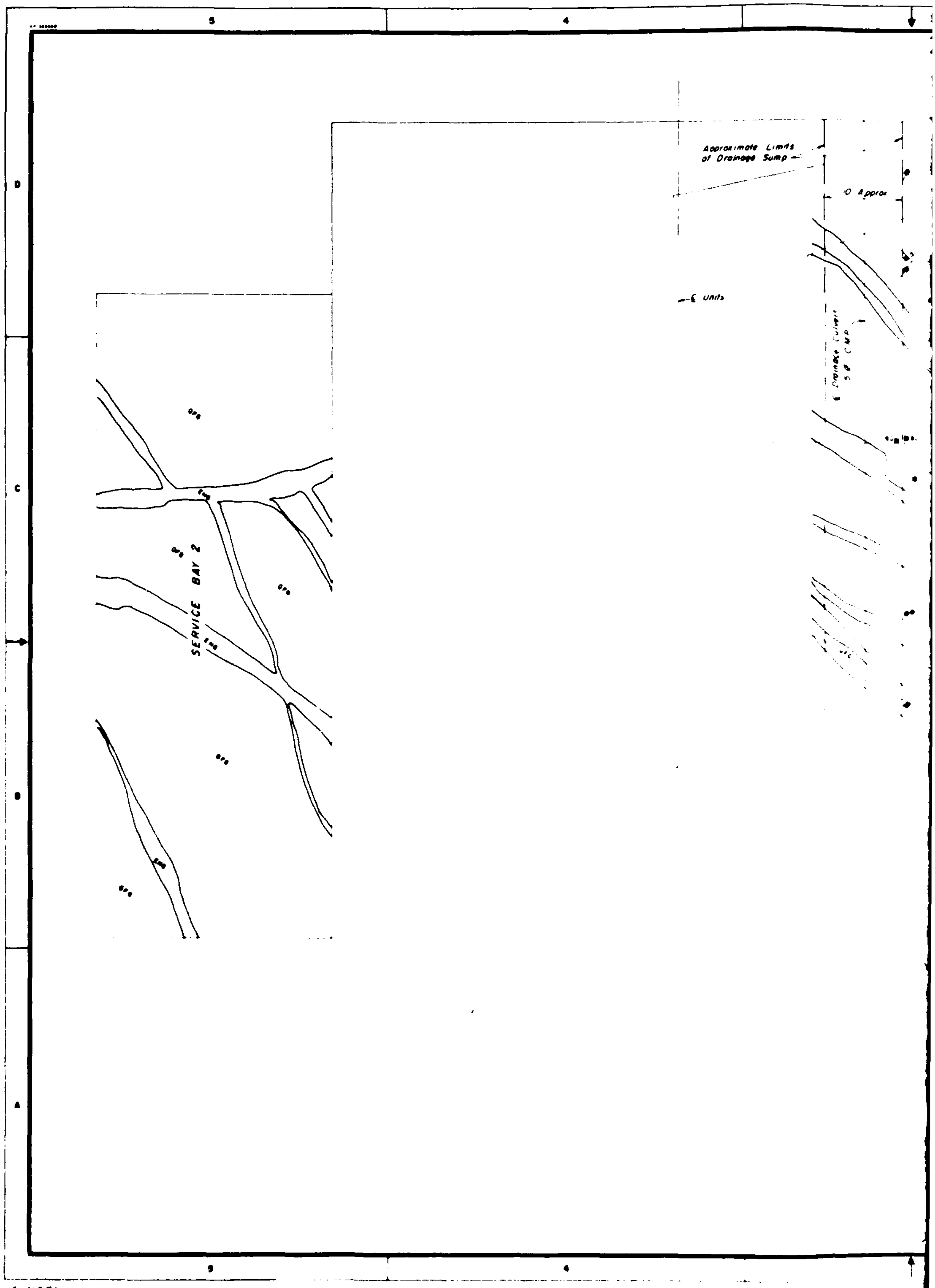


5
4
D
C
B
A
Approximate Limits
of Drainage Sump
SERVICE BAY I
EHG PORPHYRY

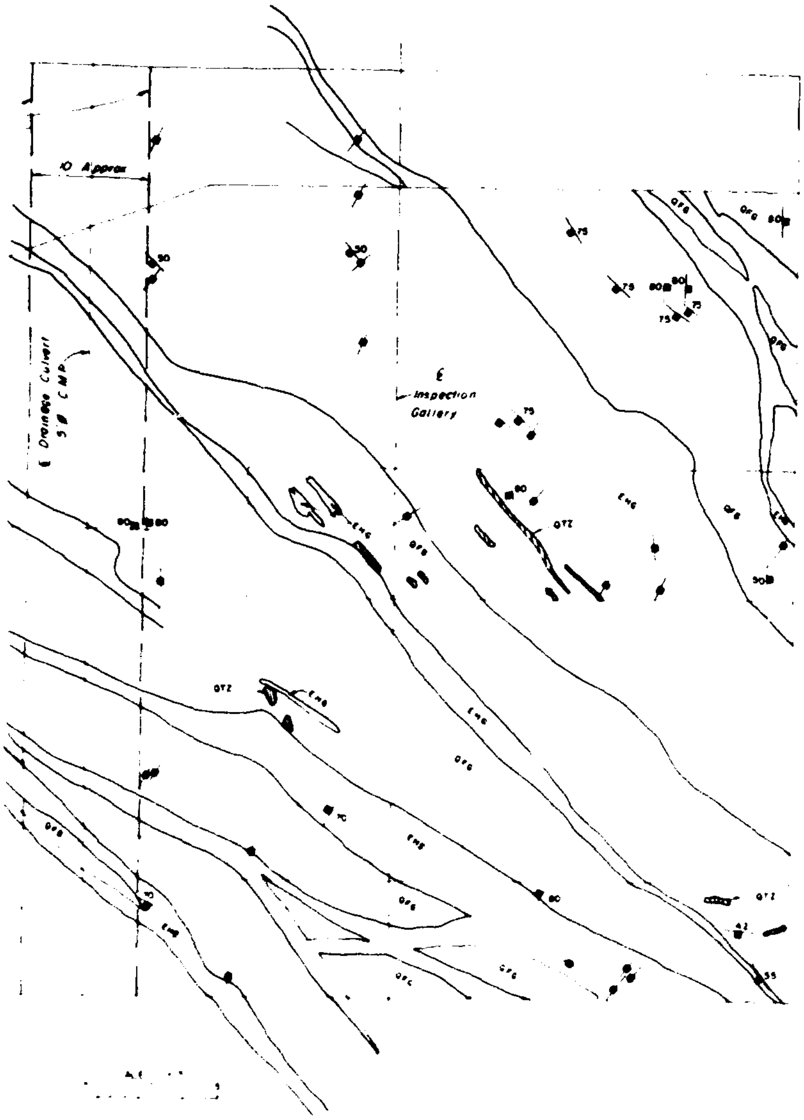


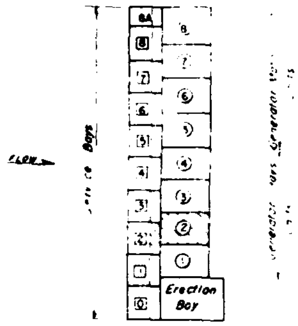
EHG PORPHYRY
EHG
QFG
EPIDOTE
QTZ
0' Approx
Inspection Gallery
EHG 2' wide
FLOW
Service Bays
Generator Bays Units
Generator Motor Bays Units
Erection Bay
LEGEND
EHG EPIDOTE HORNBLENDE GNEISS
QFG QUARTZ FELDSPAR GNEISS
QTZ QUARTZ VEIN
VERTICAL JOINT
SHEARED
SCALE
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA
POWERHOUSE
POWERHOUSE FOUNDATION MAP
UNIT-1
SAVANNAH RIVER GEORGIA AND
REVISIONS

2



5
4
D
C
B
A
Approximate Limits
of Drainage Sump
SERVICE BAY 2
Units





NOTE
For legend see Plate 120

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION
POWERHOUSE

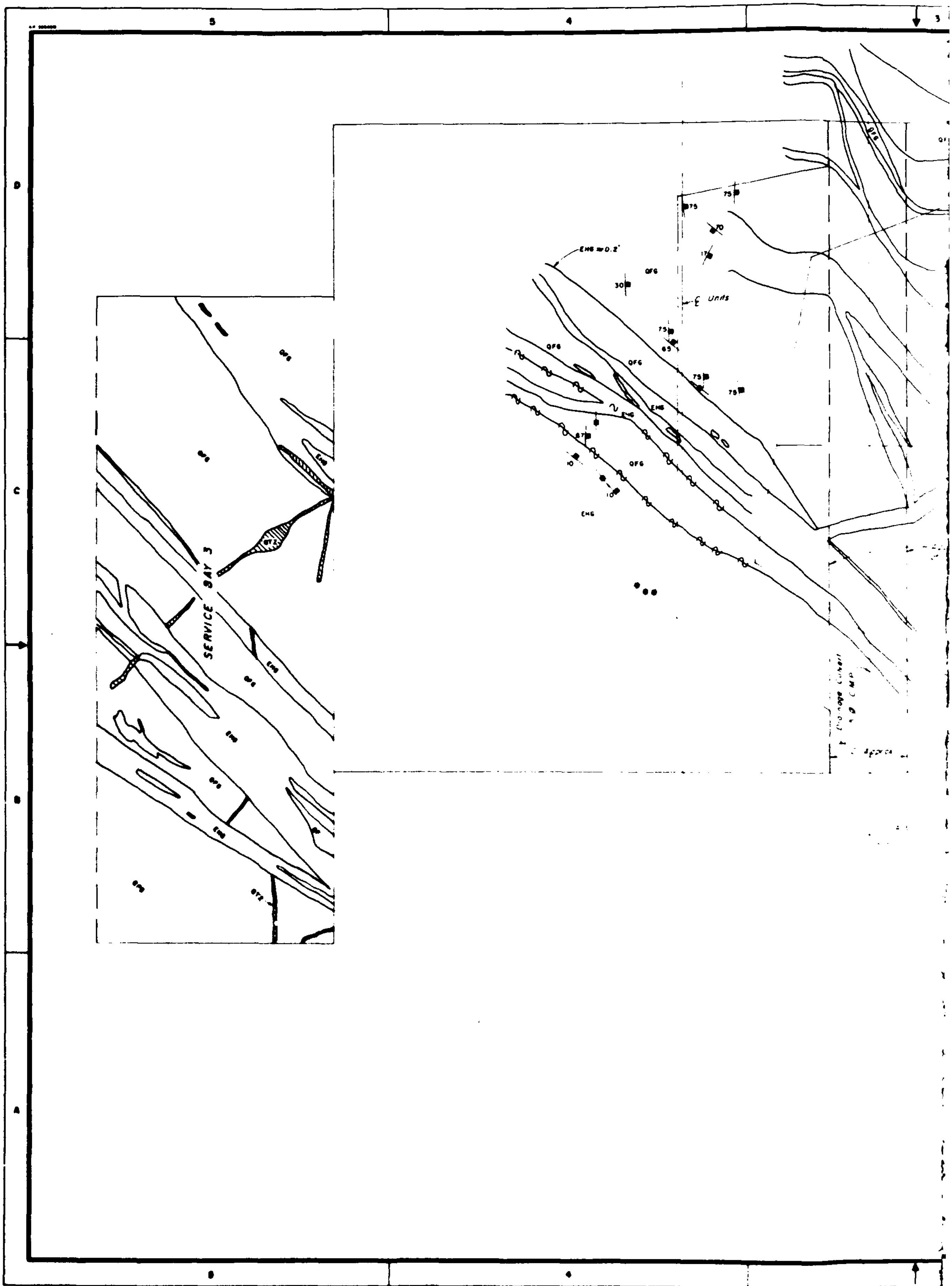
POWERHOUSE FOUNDATION MAP
UNIT - 2

RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

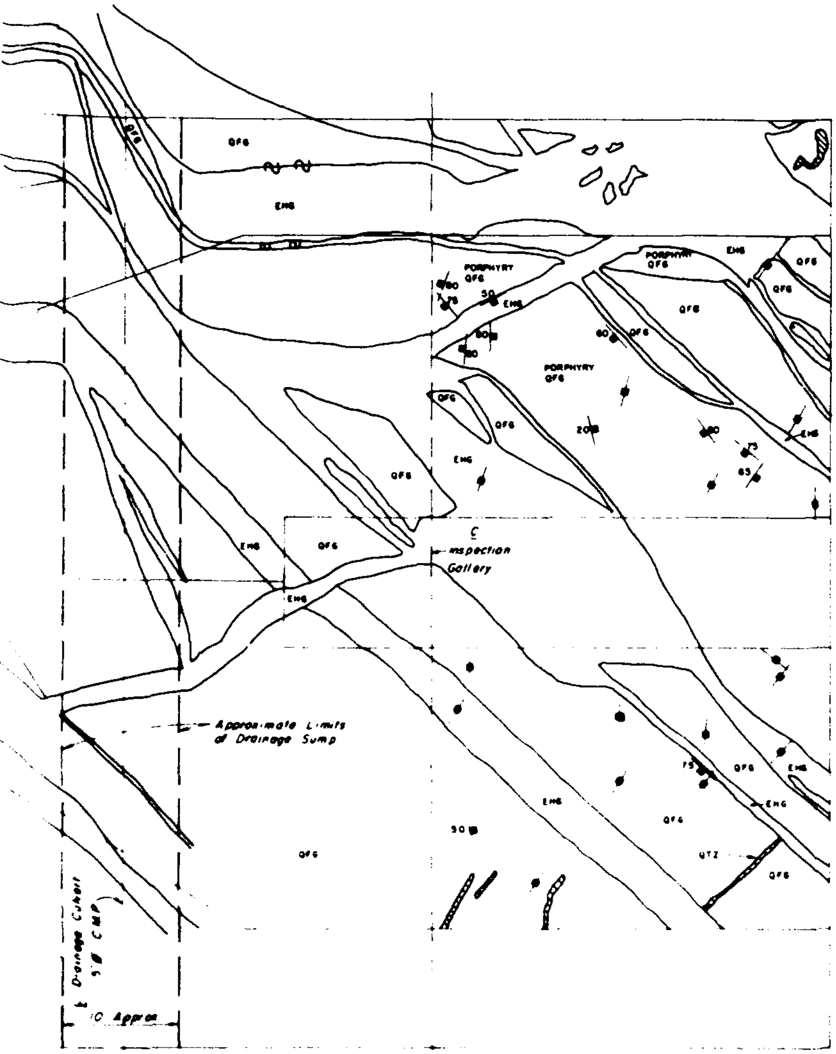
PLATE 121

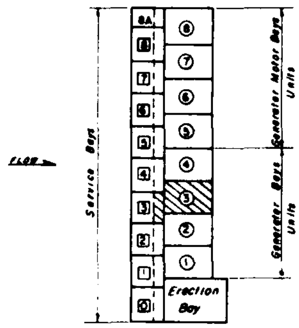
REVISIONS

2



5
4
3
D
C
B
A
SERVICE BAY 3
QFG
EMG
Units





FLOW

NOTE:
For legend see Plate 120

SCALE

SYMBOL	ZONE	DESCRIPTION	DATE	BY
REVISIONS				

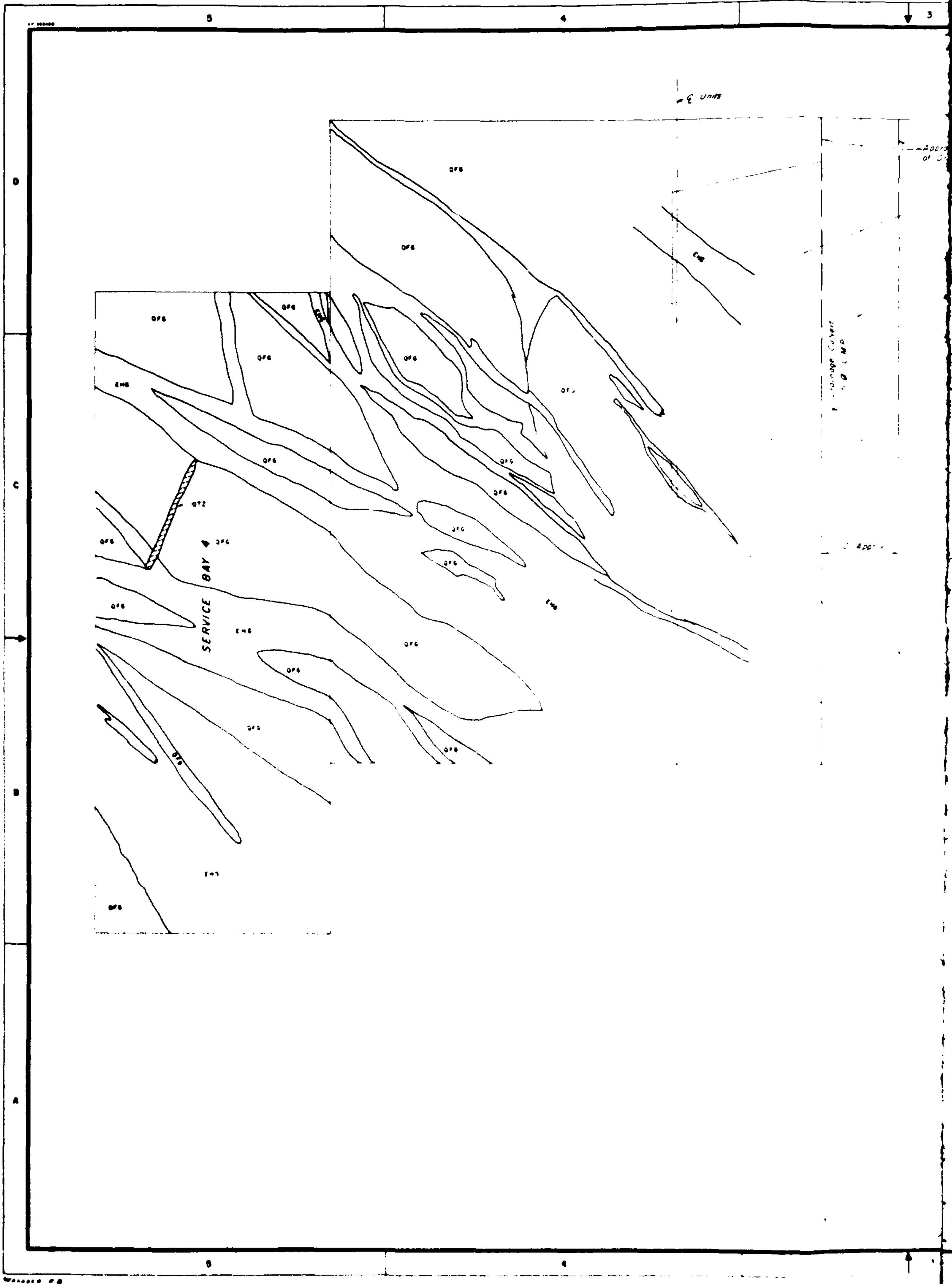
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
POWERHOUSE
POWERHOUSE FOUNDATION MAP
UNIT-3
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

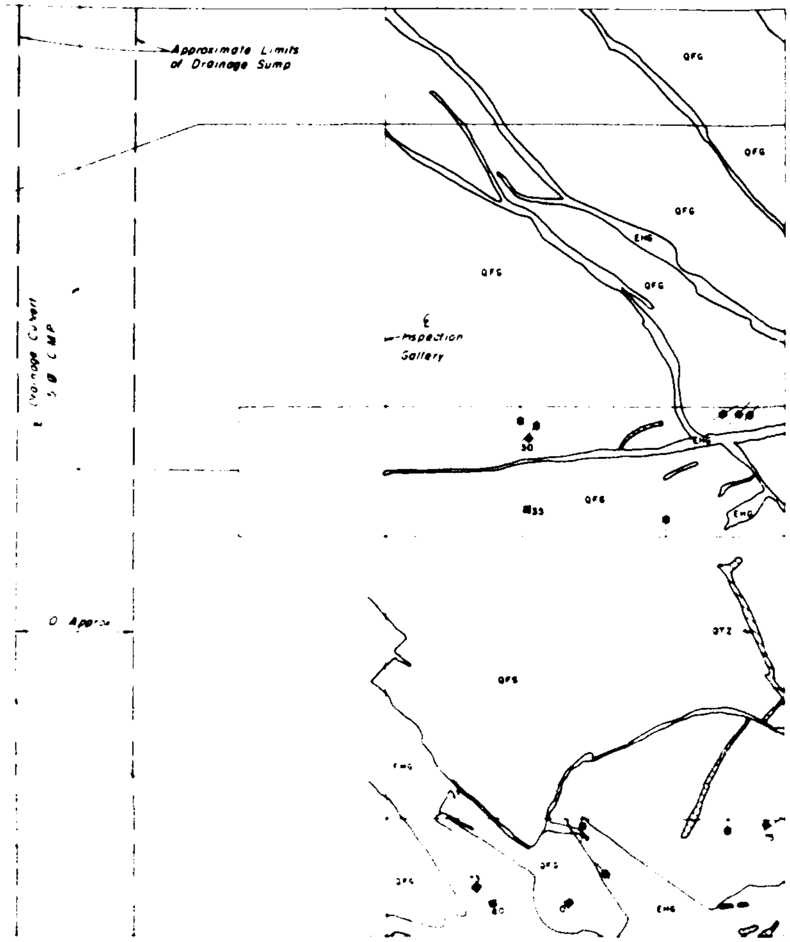
SIZE	INVITATION NO	FILE NO	PLATE
F			122

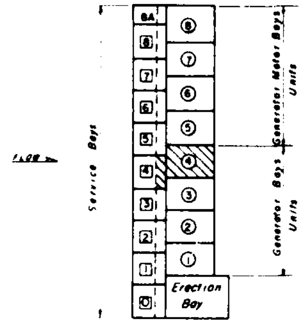
SCALE AS SHOWN | SHEET

122



5
4
3
D
C
B
A
℄ Units
QFG
EHG
QTZ
SERVICE BAY





NOTE.
For legend see Plate 120

U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
POWERHOUSE

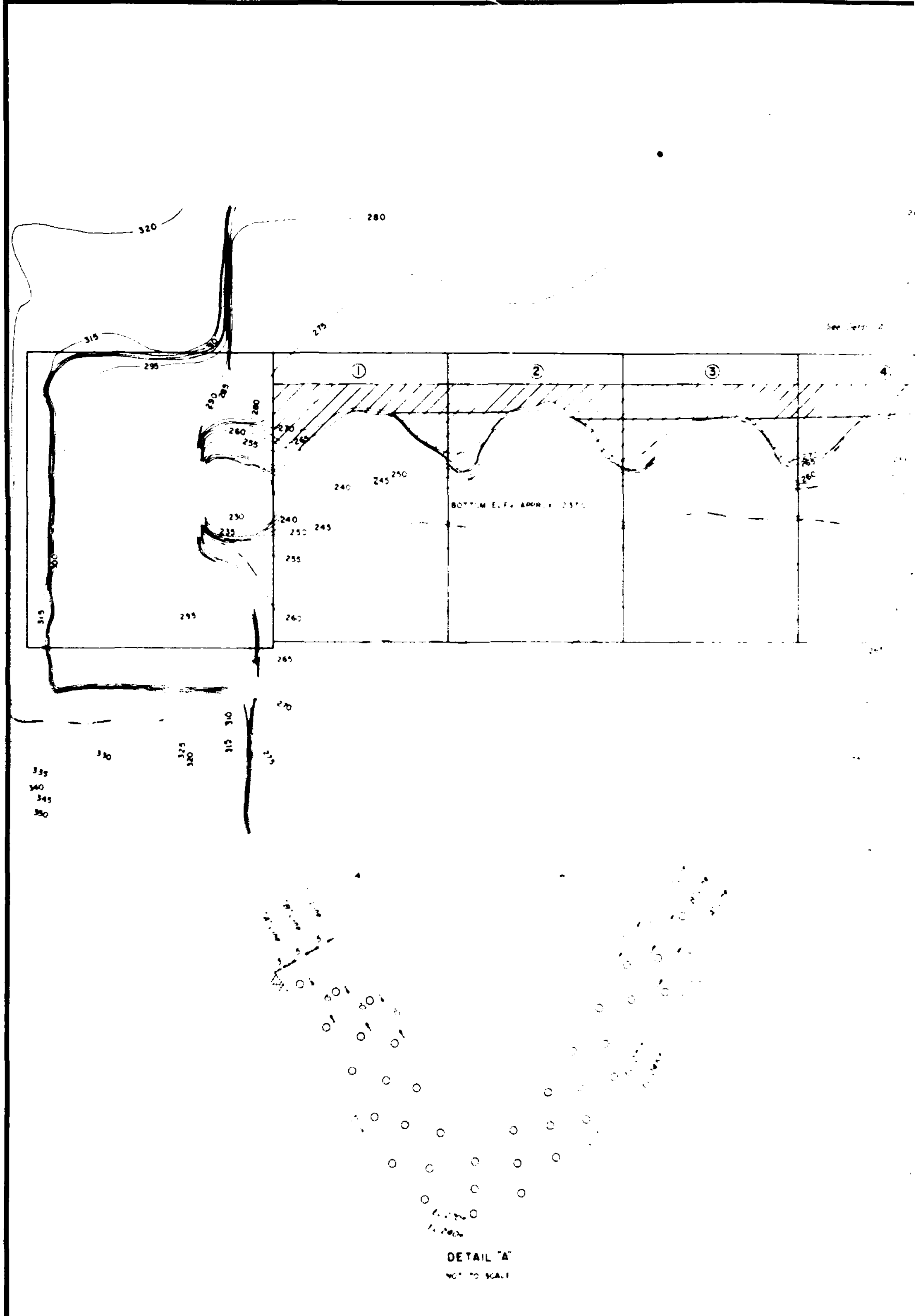
POWERHOUSE FOUNDATION MAP
UNIT - 4

RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

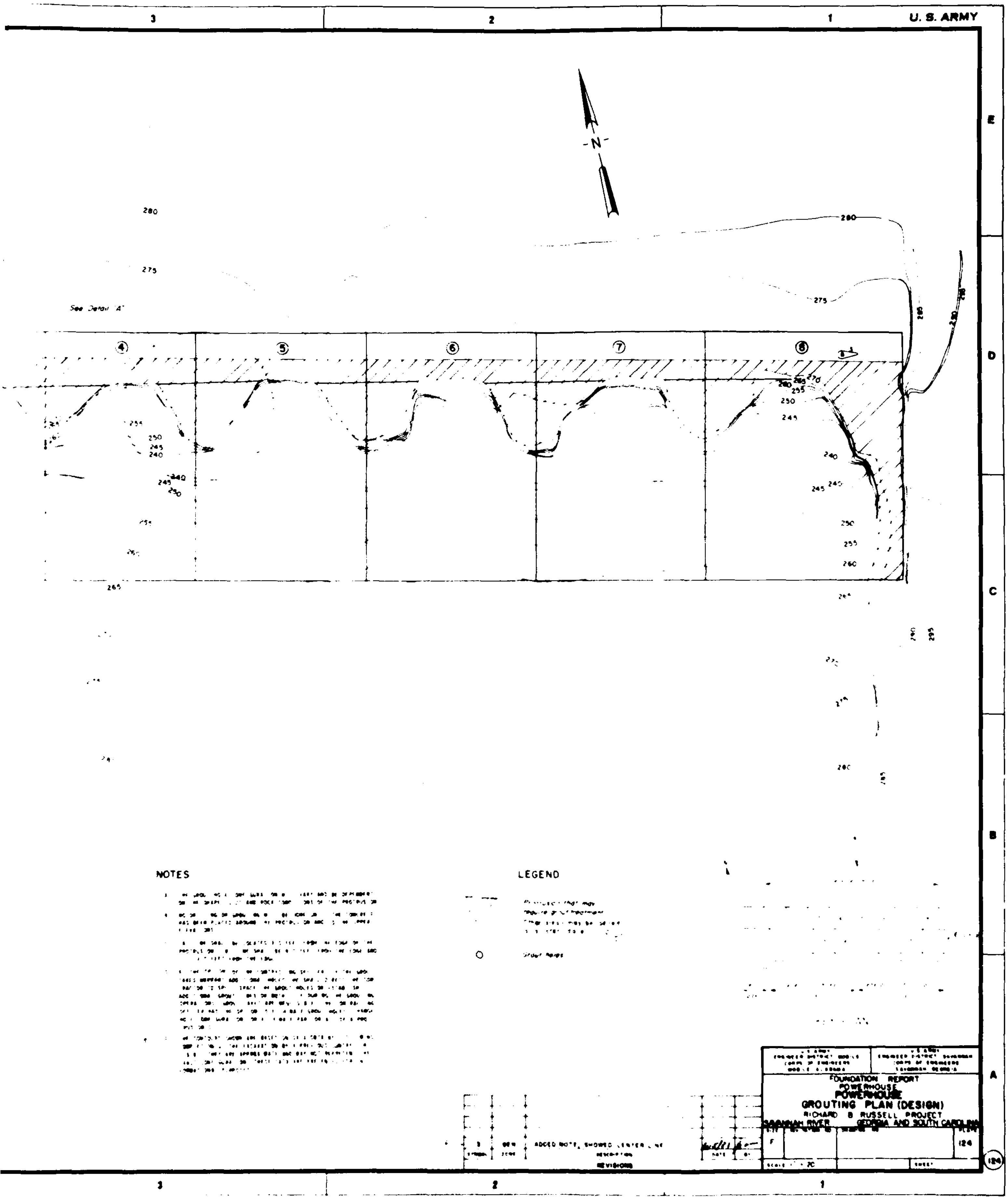
SIZE F
PLATE 123
SCALE AS SHOWN

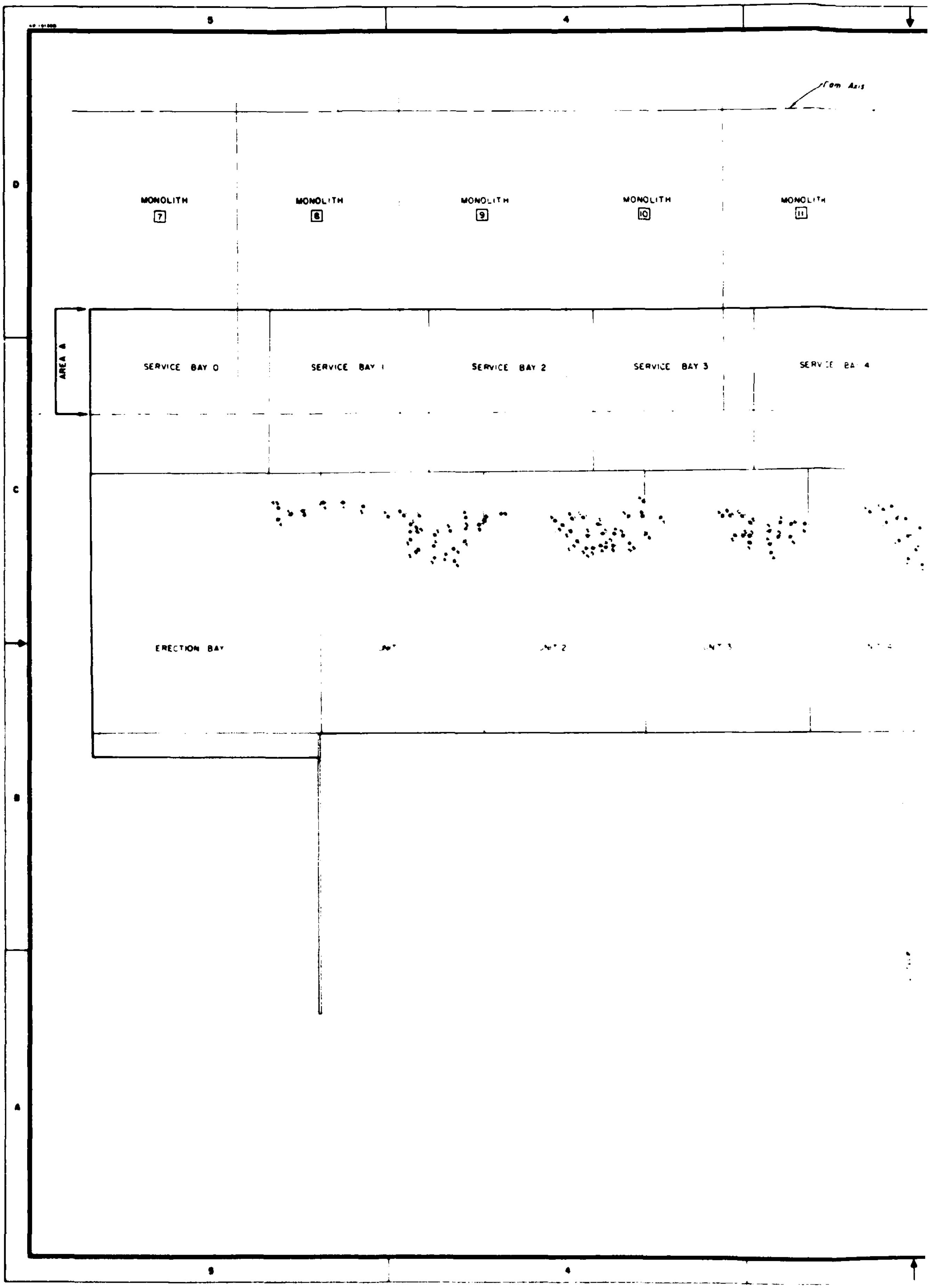
REVISIONS

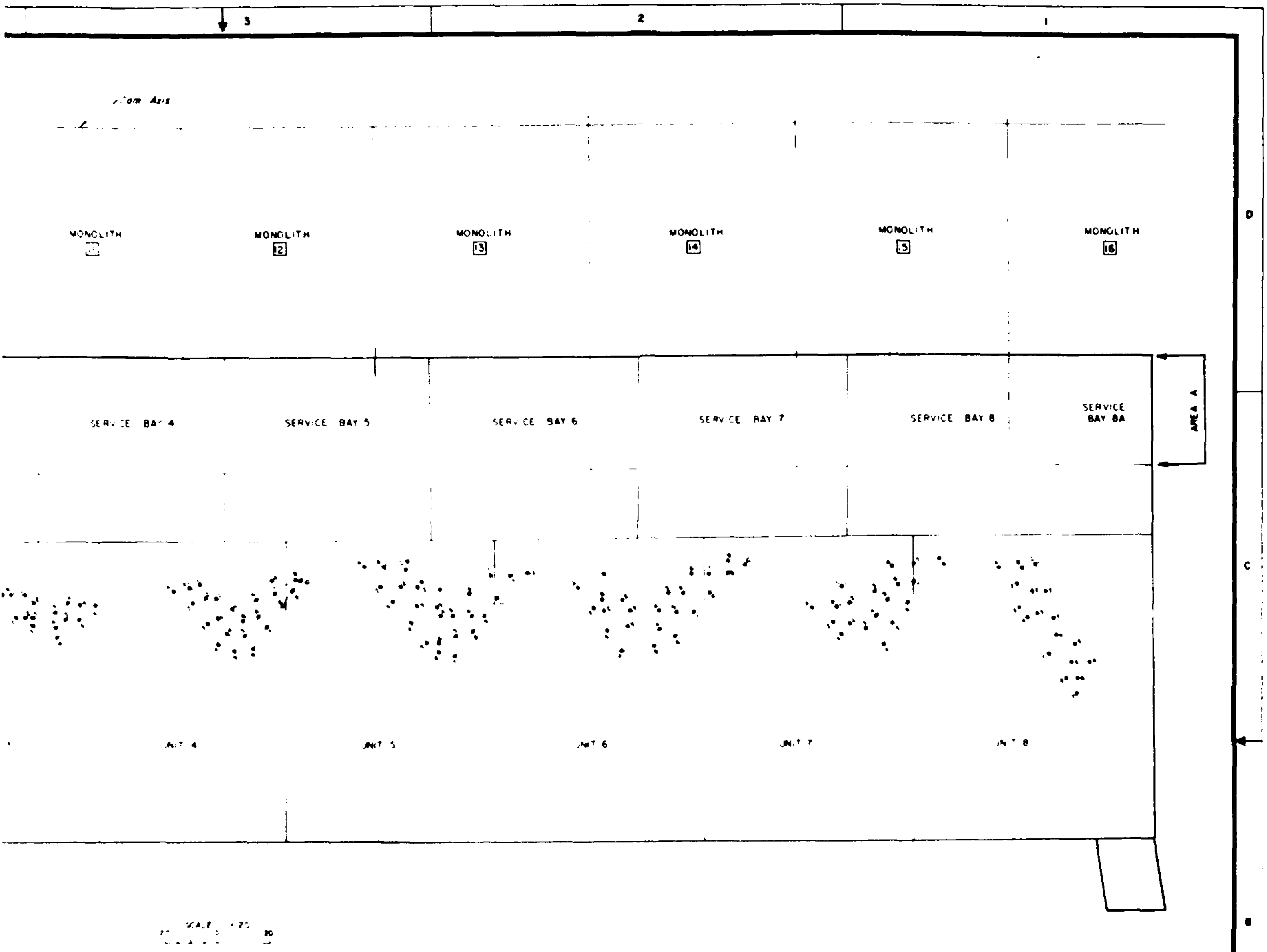
CORPS OF ENGINEERS | 4



5 | 4


U. S. ARMY
N
280
275
See Detail "A"
4
5
6
7
8
250
245
240
255
260
265
270
285
290
295
NOTES
LEGEND
Grout holes
ADDED NOTE, SHOWED CENTER LINE
DESCRIPTION
REVISIONS
FOUNDATION REPORT
POWERHOUSE
POWERHOUSE
GROUTING PLAN (DESIGN)
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA
124
SHEET
124


MONOLITH 7
MONOLITH 8
MONOLITH 9
MONOLITH 10
MONOLITH 11
AREA A
SERVICE BAY 0
SERVICE BAY 1
SERVICE BAY 2
SERVICE BAY 3
ERECTION BAY



NOTES

1. Symbol • denotes grout hole and hole number.
2. Grouting was performed in one stage.
3. All holes are approximately 30' deep.
4. Area A is part of the concrete dam excavation contract. Refer to Monoliths 8 thru 6, Plates 26 thru 34 for geologic and topographic information.
5. Grout hole information is summarized on Table 12.

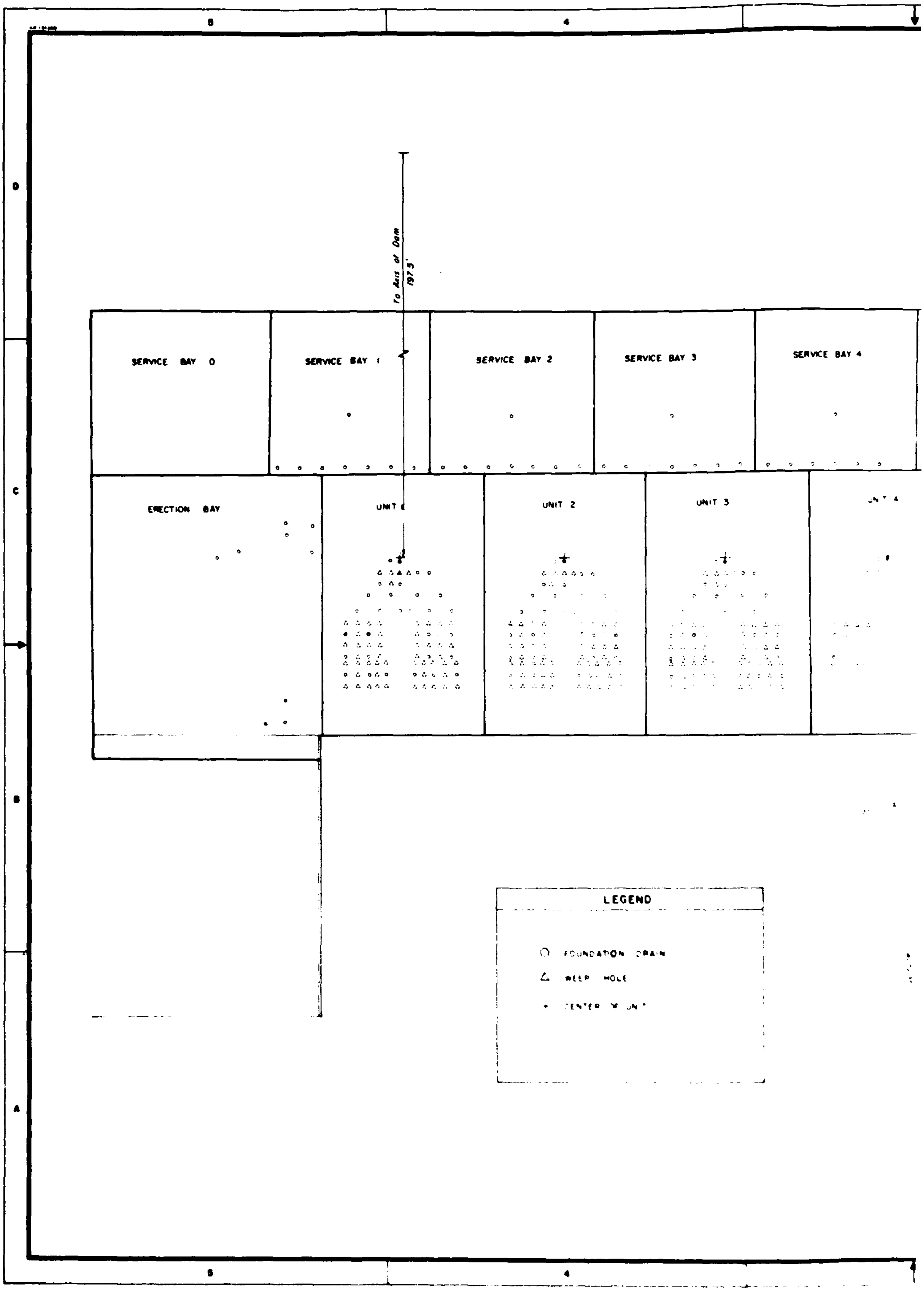
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
POWERHOUSE
POWERHOUSE FOUNDATION
GROUTING (AS-BUILT)
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

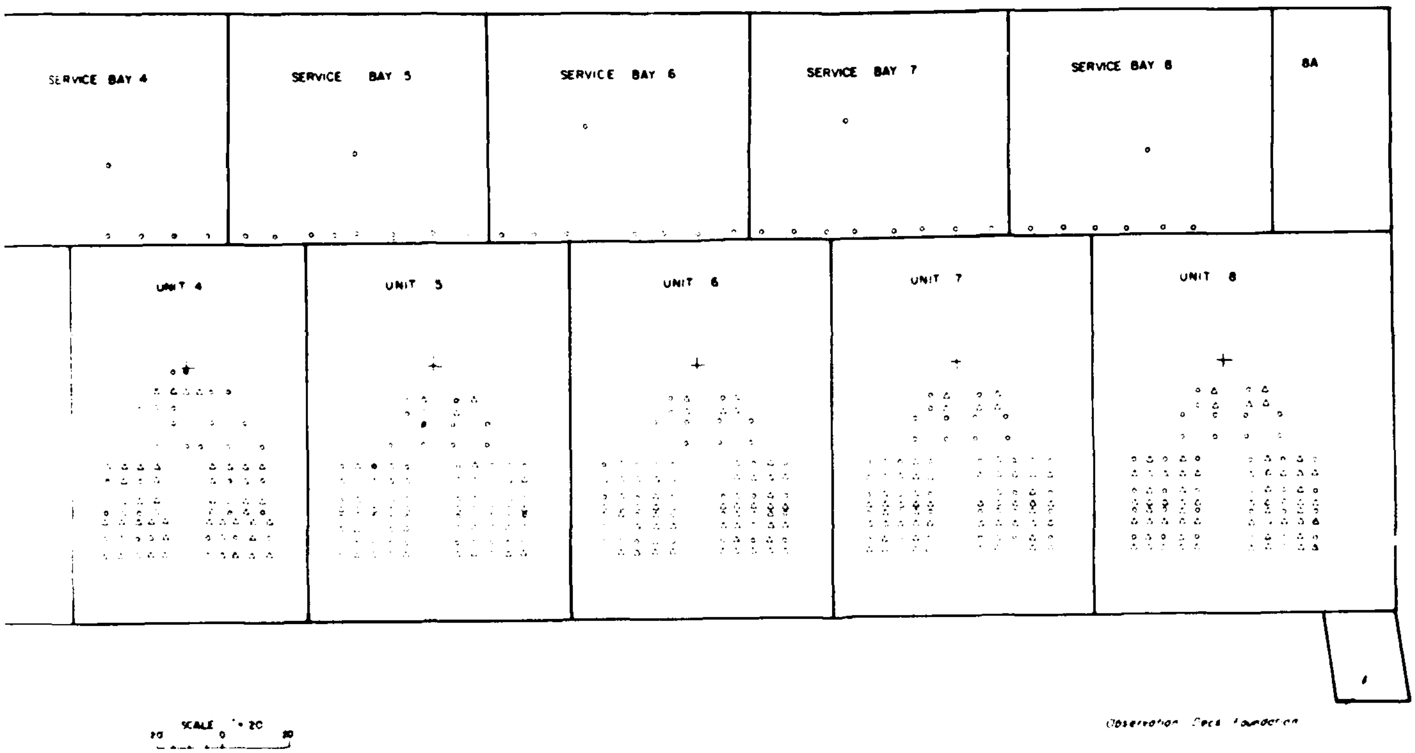
PLATE 128

SCALE AS SHOWN

REVISIONS



To Axis of Dam
197.5'
SERVICE BAY 0
SERVICE BAY 1
SERVICE BAY 2
SERVICE BAY 3
SERVICE BAY 4
ERECTION BAY
UNIT 2
UNIT 3
LEGEND
FOUNDATION DRAIN
WEEP HOLE



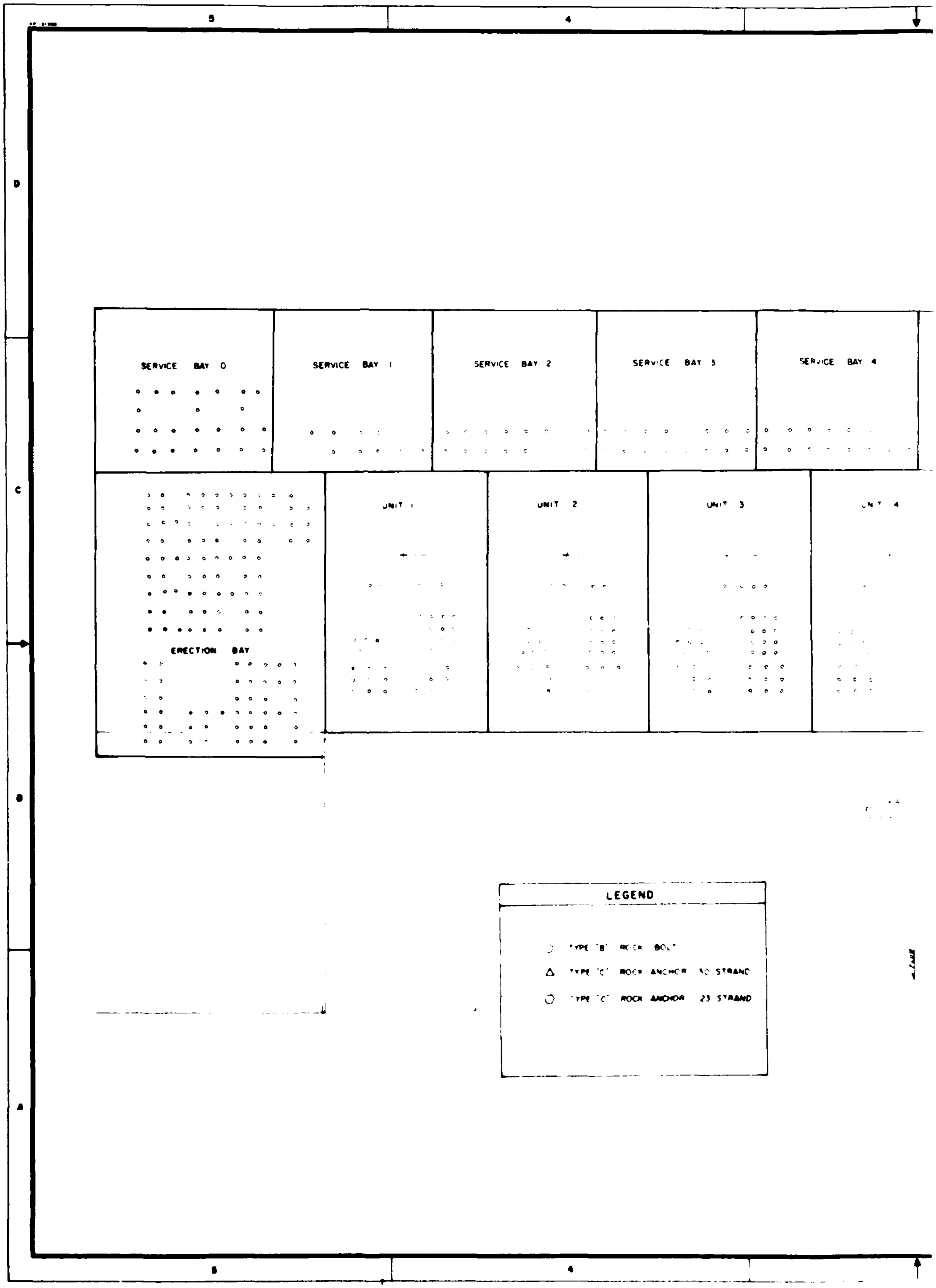
NOTE
For details of foundation drain and weep holes in draft tube area, see Plate 50

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

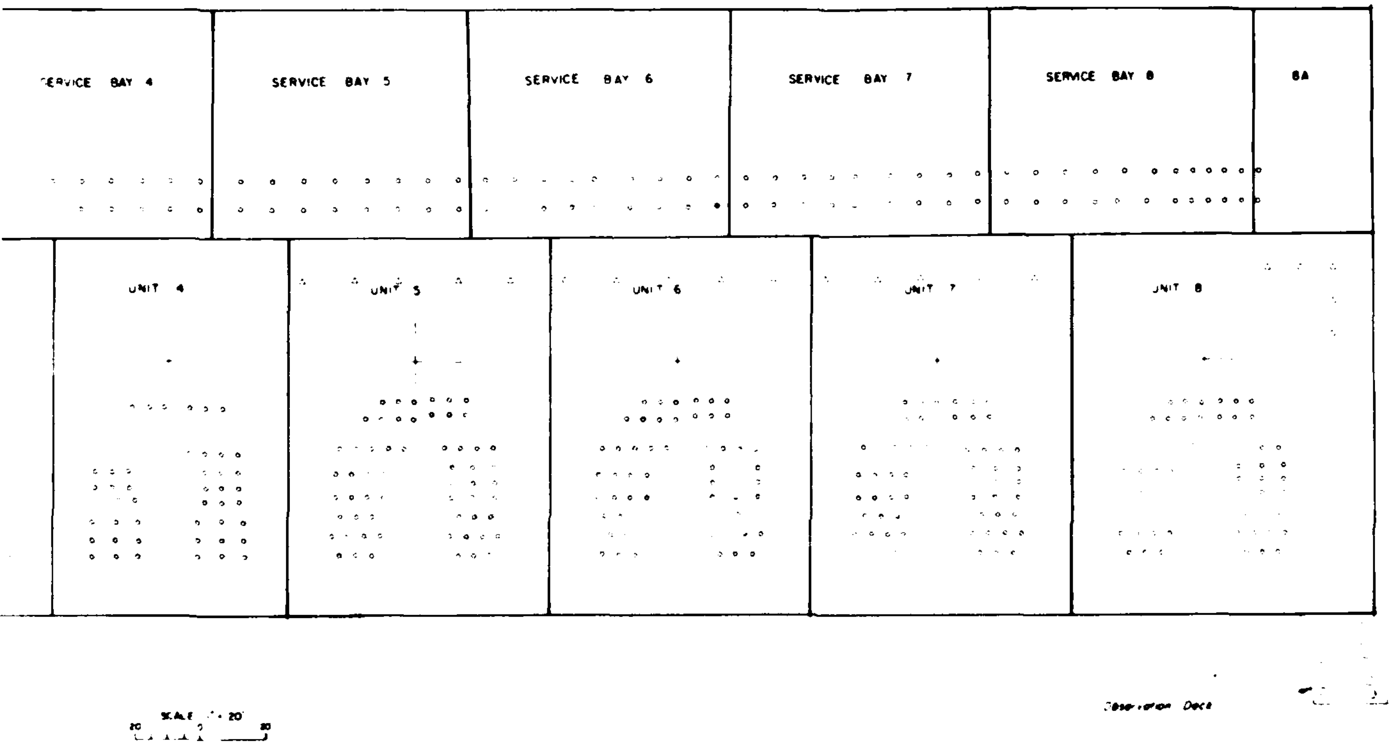
FOUNDATION REPORT
POWERHOUSE FOUNDATION
DRAINAGE SYSTEM
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 126

126



SERVICE BAY 0
SERVICE BAY 1
SERVICE BAY 2
SERVICE BAY 3
SERVICE BAY 4
UNIT 1
UNIT 2
UNIT 3
UNIT 4
ERECTION BAY
LEGEND
TYPE "C" ROCK ANCHOR 23 STRAND



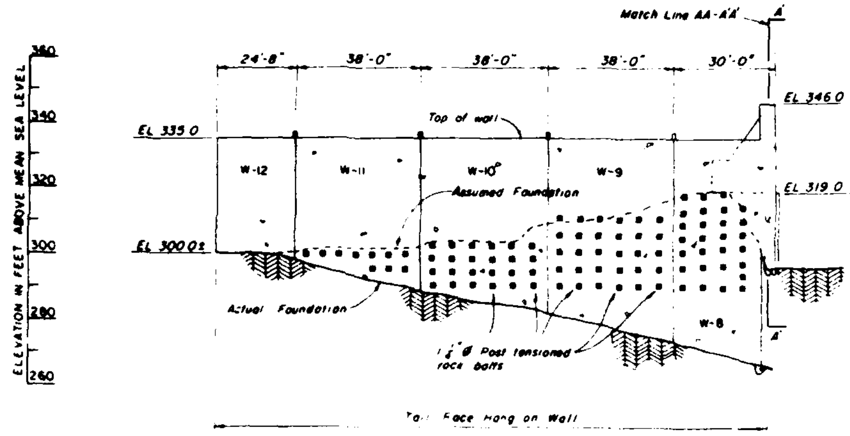
U S ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
POWERHOUSE
LOCATION OF POWERHOUSE FOUNDATION
ROCK BOLTS AND ROCK ANCHORS
RICHARD B RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

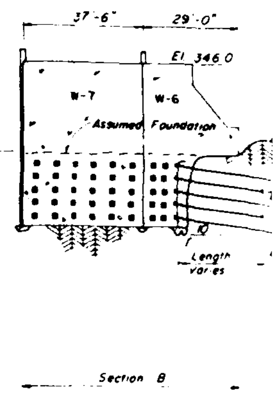
SIZE			PLATE
F			127
SCALE AS SHOWN		SHEET	

REVISIONS

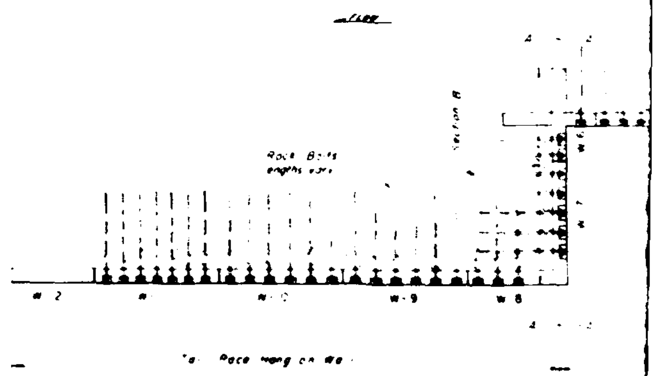
127



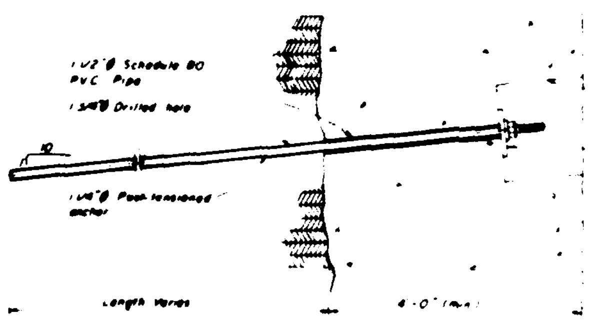
PROFILE



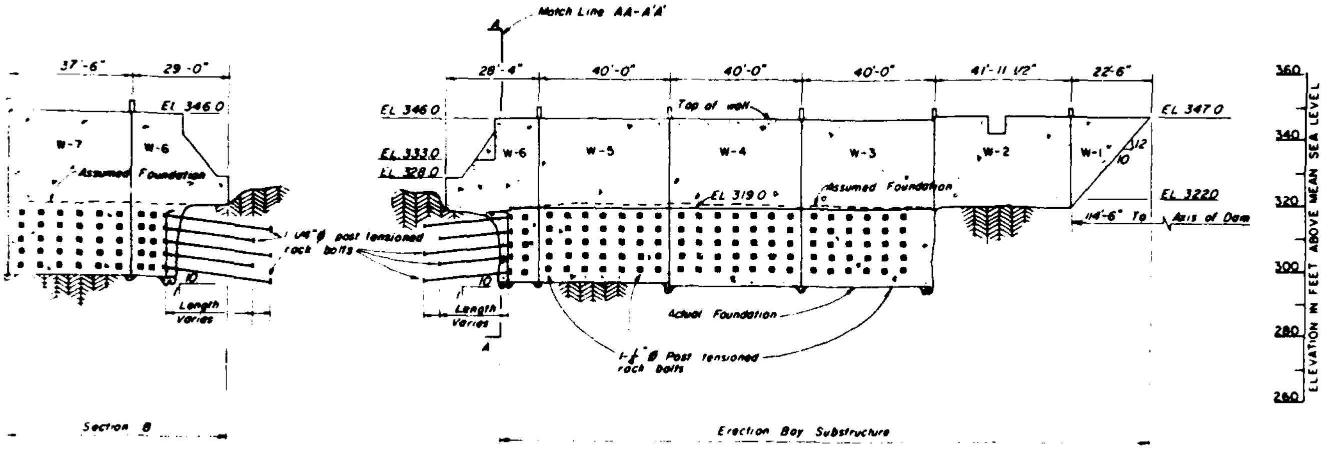
PROFILE



PLAN

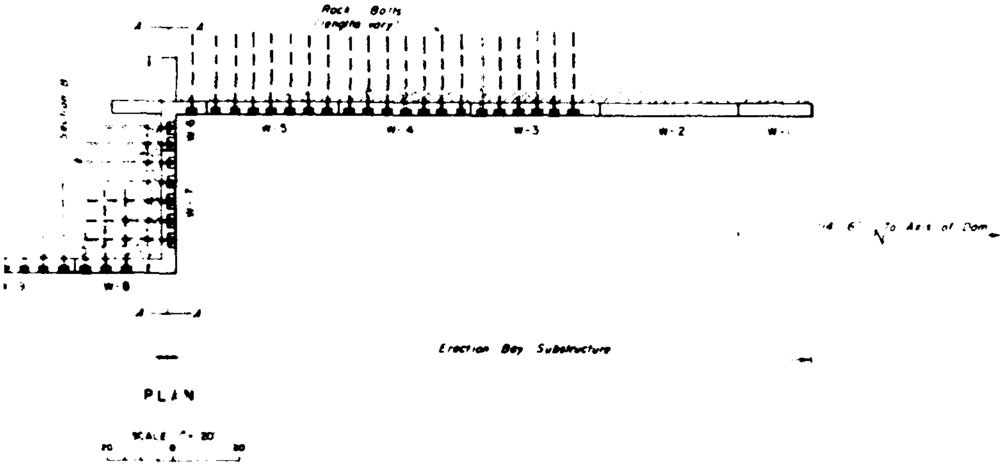


TYPE A ROCK BOLT INSTALLATION (TYPICAL)



PROFILE

PROFILE



PLAN

LEGEND

SOUND ROCK (PROFILE)

SOUND ROCK (PLAN VIEW)

ROCK BOLTS

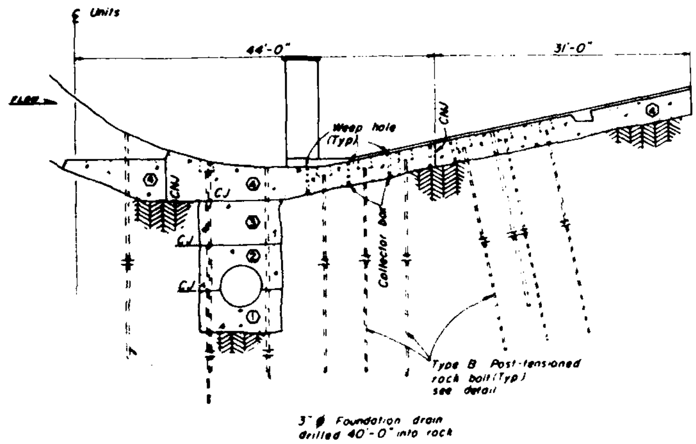
U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT-POWERHOUSE
TAIL RACE HANG-ON WALL & ERECTION BAY SUBSTRUCTURE
ROCK BOLT PATTERN & DETAILS
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

PLATE 128

REVISIONS

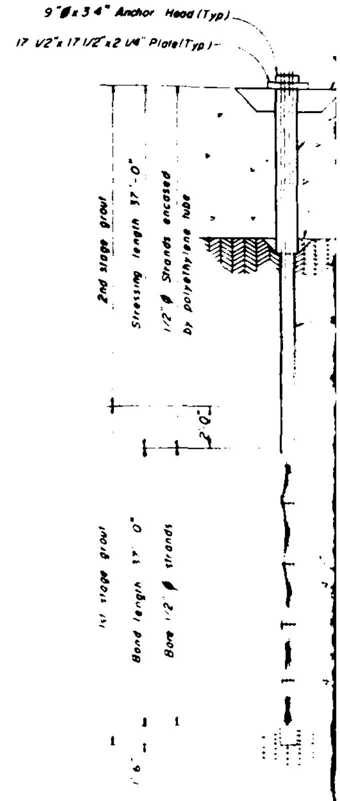
128



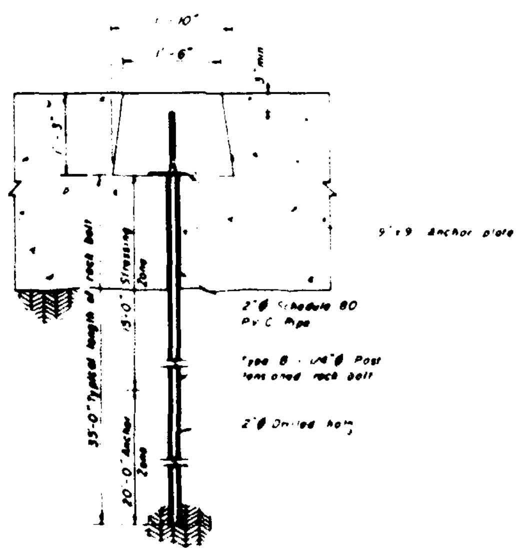
TYPICAL SECTION THRU DRAFT TUBE AREA

SCALE 1/8" = 1'-0"

C.J. = Construction Joint
CNJ = Contraction Joint
⑤ Concrete Lift

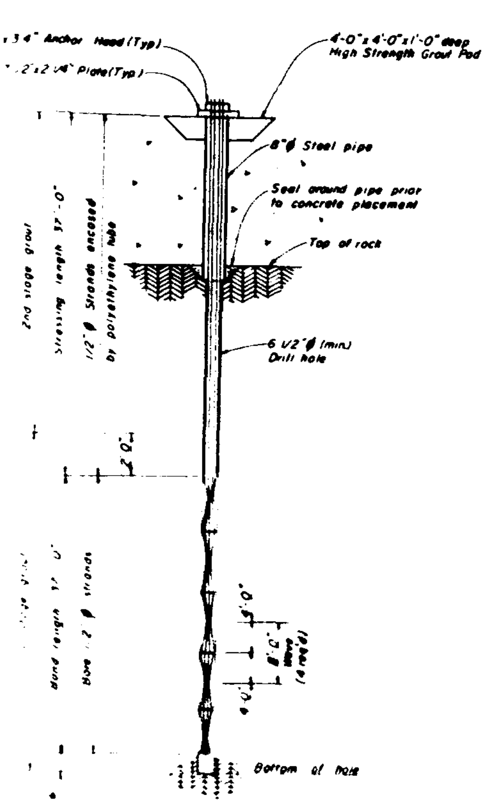


TYPE C-30 STRAND ROCK A



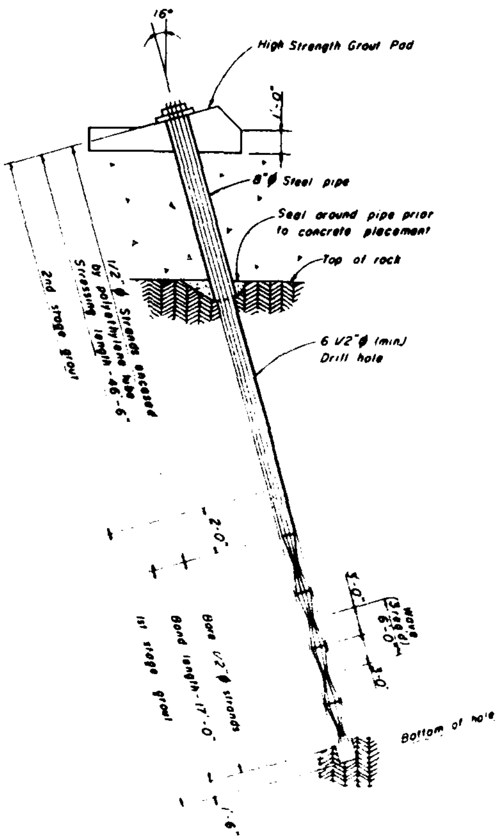
TYPE B-ROCK BOLT INSTALLATION (TYP)

SCALE 1" = 1'-0"



TYPE C 30 STRAND ROCK ANCHOR (TYP)

NOT TO SCALE



TYPE C 23 STRAND ROCK ANCHOR (TYP)

NOT TO SCALE

U. S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

FOUNDATION REPORT
POWERHOUSE
DETAILS - ROCK BOLTS
ROCK ANCHORS
RICHARD B. RUSSELL PROJECT
SAVANNAH RIVER GEORGIA AND SOUTH CAROLINA

F			PLATE 129
SCALE AS SHOWN		SHEET	

REVISIONS

129

2